PILOT SCALE OPTIMIZATION OF BIOLOGICAL NITROGEN REMOVAL FROM DOMESTIC WASTEWATER FROM SELECTED COMMUNITIES IN EASTERN PROVINCE

BY

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To my parents, my siblings, and my best friends who supported me during this journey.

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LIST OF ABBREVIATIONS

Constituent	Definition
BOD	Total 5-d biochemical oxygen demand, mg/*l*
sBOD	Soluble 5-d biochemical oxygen demand, mg/*l*
UBOD	Ultimate biochemical oxygen demand, mg/*l*
COD	Total chemical oxygen demand, mg/*l*
bCOD	Biodegradable chemical oxygen demand, mg/*l*
pCOD	Particulate chemical oxygen demand, mg/*l*
sCOD	Soluble chemical oxygen demand, mg/*l*
nbCOD	Nonbiodegradable chemical oxygen demand, mg/*l*
rbCOD	Readily biodegradable chemical oxygen demand, mg/*l*
bsCOD	Biodegradable soluble chemical oxygen demand, mg/*l*
sbCOD	Slowly biodegradable chemical oxygen demand, mg/*l*
bpCOD	Biodegradable particulate chemical oxygen demand, mg/*l*
nbpCOD	Nonbiodegradable particulate chemical oxygen demand, mg/*l*
nbsCOD	Nonbiodegradable soluble chemical oxygen demand, mg/*l*
TKN	Total Kjeldhal nitrogen, mg/*l*
bTKN	Biodegradble total Kjeldhal nitrogen, mg/*l*
TKN_o	Initial Total Kjeldal nitrogen, mg/*l*
ON	Soluble filtered total Kjeldhal nitrogen, mg/*l*
bON	Biodegradable organic nitrogen, mg/*l*
nbON	Nonbiodegradable organic nitrogen, mg/*l*
pON	Particulate organic nitrogen, mg/*l*
nbpON	Nonbiodegradable particulate organic nitrogen, mg/*l*

sON	Soluble organic nitrogen, mg/*l*
nbsON	Nonbiodegradable soluble organic nitrogen, mg/*l*
pH	potential of hydrogen
TSS	Total suspended solids, mg/*l*
VSS	Volatile suspended solids, mg/*l*
nbVSS	Nonbidegradable volatile suspended solids, mg/*l*
iTSS	Inert total suspended solids, mg/*l*
SBR	Sequencing batch reactor
Ne	Nitrogen in the effluent, mg/*l*
NH_4-N	Ammonia nitrogen, mg/*l*
TP	Total phosphorus, mg/*l*
SVI	Sludge volume index
NOx	Concentration of NH_4-N in the influent flow that is nitrified, mg/*l*
tA	Time of aeration, hours
tS	Time of settling, hours
tD	Time of decanting, hours
tF	Time of filling, hours
B32*	Second batch of the third sample
Q	Flow rate, m^3/d
V_T	Total volume of the tank, m^3
V_S	Settled volume after decanting, m^3
X	MLSS concentration at full volume, mg/*l*
X_S	MLSS concentration in settled volume, mg/*l*
V_F	Volume of filling, m^3
HRT	Hydraulic retention time, hours
SRT	Solids retention time, days

$\mathbf{P_{X,TSS}}$	Net waste activated sludge produced each day measured in terms of total suspended solids, kg/d
$\mathbf{S_o}$	Influent concentration, mg/*l*
S	Concentration of growth-limiting substrate in solution, mg/*l*
$\mathbf{P_{X,VSS}}$	Net waste activated sludge produced each day measured in terms of volatile suspended solids, kg/d
$\mathbf{P_{X,bio}}$	Biomass as VSS wasted, g/d
$\mathbf{N_o}$	NH_4-N concentration at time, mg/*l*
$\mathbf{N_t}$	NH_4-N concentration at time t, mg/*l*
$\mathbf{X_n}$	Nitrifying bacteria concentration, mg/*l*
$\mathbf{R_o}$	Total oxygen requirement, kg/d
F/M	Food to microorganism ration, g/g.d
$\mathbf{L_{org}}$	BOD volumetric loading kg/m^3.d
$\mathbf{NO_3}$**-N**	Nitrate, mg/*l*
$\mathbf{NO_r}$	Nitrate removed, g/d
SDNR	Specific denitrification rate, g NO_3-N/g MLVSS.d
S3**	The third sample

* The first number is for sample number, and the second number is for batch number.

** The (S) refers to sample, and the number refers to the number of the sample.

THESIS ABSTRACT

FULL NAME : Ahmed Mohammed Khalafalla Allazem

TITLE OF STUDY : Pilot Scale Optimization of Biological Nitrogen Removal from Domestic Wastewater from Selected Communities in Eastern Province

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The overall aim of this study was to investigate the performance of the sequencing batch reactor (SBR) in the treatment of municipal wastewater, particularly, the removal of nitrogen. SBR is mainly used for the treatment of industrial wastewater as small-scale package plants. This investigation hopes to shed more light on the advantages of this process in the treatment of municipal wastewater.

The study started with the assessment of the performance of secondary treatment of North Sewage Treatment Plant (NSTP) in Dhahran, Saudi Arabia where the influent wastewater for the study was taken. The rational for the assessment is to enable this study to compare with the results of this existing plant. Four months of operation data were obtained by analyzing NSTP Influent and effluent samples. The characterization of the municipal wastewater (WW) consisted of pH, Turbidity, Total Suspended Solids (TSS) chemical oxygen demand (COD), biochemical oxygen demand in 5 days (BOD_5), Nitrate, and Ammonia. The assessment showed that the existing plant is performing well, and all

the parameters tested were within the standards set by the regulating authority (Ministry of Environment, Water and Agriculture).

After the assessment was completed, a pilot scale SBR was used for the experimental study. The parameters tested in this study included; pH, turbidity, TSS, COD and the biological nitrogen removal. Two scenarios were performed, pre-anoxic and post-anoxic to study removal of nitrogen. The results showed that post-anoxic denitrification was more efficient in the removal of Total nitrogen (50%) than pre-anoxic denitrification (21.7%). However, for the removal of Total Kjeldhal nitrogen the removal efficiency was almost the same around 94%. For Ammonia removal, the two scenarios gave similar results around 92% removal. As far as other parameters were concerned, similar results were obtained from the two scenarios. COD removal efficiency reached around 73%, turbidity averaged around 96%, and Total Suspended Solids averaged around 98%.

In comparison with the existing NSTP, the removal efficiencies of all parameters tested for the SBR process compares very well. This shows that the SBR process performs well in treating municipal wastewater.

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ملخص الرسالة

الإسم بالكامل : **أحمد محمد خلف الله اللازم**

عنوان الرسالة: **الطريقة الأمثل لإزالة النيتروجين بيولوجياً من مياه الصرف الصحي المنزلية بإستخدام محطة تجريبية.**

التخصص: **الهندسة المدنية والبيئية**

تاريخ الدرجة العلمية: **ربيع الثاني 1439**

الهدف الرئيسي من هذا البحث هو دراسة إداء المفاعل ذو الدفعات المتسلسلة في معالجة مياه الصرف الصحي المحلية، وتحديداً إزالة النتروجين. المفاعل ذو الدفعات المتسلسلة يستخدم بصورة أساسية في معالجة مياه الصرف الصحي الصناعية كوحدة معالجة صغيرة الحجم. في هذا البحث نأمل أن نسلط الضوء على فوائد هذه الطريقة في معالجة مياه الصرف الصحي المحلية.

بدأت الدراسة بعمل تقييم لأداء محطة معالجة مياه صرف صحي محلية تعرف بإسم المحطة الشمالية لمعالجة مياه الصرف والموجودة بالدوحة – الظهران (المملكة العربية السعودية) وهي المحطة التي أخذت منها مياه الصرف الصحي لهذه الدراسة. الفكرة من هذا التقييم هو تمكين هذه الدراسة من مقارنة نتائجها مع المحطة الموجودة فعلياً.

قمنا بتحليل عينات مياه الصرف الصحي لمدة أربعة شهور متواصلة. تحليل العينات تضمن (الرقم الهديدروجيني، العكارة، المواد الصلبة العالقة، الحمل البيولوجي، الحمل الكيميائي، النترات و الأمونيا). أظهر التقييم أن المحطة تؤدي المهمة التي صممت من أجلها بطريقة فعالة، وأن كل المعاملات التي أختبرت كانت موافقة للمعاير التي وضعتها السلطات المنظمة وهي وزارة البيئة والمياه والزراعة.

بعد أن أجري هذا التقييم، تم استخدام محطة تجريبية للمفاعل ذو الدفعات المتسلسلة للدراسة المعملية. المعاملات التي تم اختبارها في هذه الدراسة تضمنت أيون الهيدروجين، العكارة، الأجسام الصلبة العالقة، الحمل الكيميائي للأوكسجين و عملية إزالة النيتروجين بيولوجياً. وأجريَ ذلك عن طريقين: أولا، إزالة النتروجين بإستخدام طريقة

عوز الأكسجين بدأً (أي أن المنطقة الهوائية بعد منطقة نقص الأكسجين). والطريقة الثانية وهي نزع النتروجين بعد منطقة عوز الأكسجين. أظهرت النتائج أن طريقة إزالة النتروجين بعد منطقة عوز الأكسجين أكثر كفاءة في إزالة النتروجين (50%) من طريقة إزالة النتروجين قبل منطقة عوز الأكسجين (21.7%). بالنسبة لنتروجين خلدال فإن نسبة الإزالة كانت تقريباً متساوية في الطريقتين (94%). بالنسبة لإزالة الأمونيا فإن الطريقتين أعطيتا نتيجتين متساويتين تقرباً حوالي (92%). فيما يتعلق بالمعلمات الأخرى، تم الحصول على نتائج مماثلة بين السيناريوهين. نسبة كفاءة إزالة الحمل الكيميائي كانت (73%)، و نسبة كفاءة إزالة العكارة كانت (96%) و نسبة كفاءة إزالة الأجسام الصلبة العالقة كانت (98%).

بالمقارنة مع المحطة الشمالية لمعالجة مياه الصرف فإن نسب كفاءات الإزالة لكل العوامل المختبرة للمفاعل ذو الدفعات المتسلسلة قابلة للمقارنة بشكل واضح. هذا يظهر أن المفاعل ذو الدفعات المتسلسلة هو تقنية ممتازة لمعالجة مياه الصرف الصحي المحلية.

درجة الماجستير

جامعة الملك فهد للبترول والمعادن

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CHAPTER 1

INTRODUCTION

Wastewater treatment engineering is an essential branch of Environmental Engineering. The importance of wastewater treatment comes from the needs to protect the environment from pollution, as well as, conserve resources by reusing treated wastewater. So, the major goal for wastewater treatment is to keep the discharged treated waters (i.e. effluents) below the hazardous levels.

The wastewater contains several organic and inorganic materials as well as pathogenic organisms. The removal of such elements require a specific wastewater treatment technologies, depending on the type of wastewater (municipal, industrial …etc.) and the acceptable level of pollution in the effluents. These technologies are physical treatment operations, chemical processes, and biological treatment processes. Biological processes are the most common processes used for the treatment of municipal wastewater. Many types of biological processes are commonly used as main processes for treating municipal wastewater, including conventional activated sludge, complete mix, extended aeration, oxidation ditch, and others. Other processes such as Sequencing Batch Reactors (SBR) are used mainly for industrial wastewater [1].

The SBR process presents flexibility in the treatment of inconstant influents. It has lowest operator interaction, and it can be used for aerobic or anaerobic environments in the same chamber. Moreover it presents excellent oxygen interaction with microorganisms and

substrate and it occupies lesser ground area, and it has superior removal efficiency and it requires less energy input. These benefits validate the latest rise in the operation of this process in industrial and municipal wastewater treatment [2].

The main objective of this research is to consider the features of SBR as a flexible, compact, efficient process in studying the ability of this technology in the treatment of municipal wastewater, particularly, in nitrogen removal from municipal wastewater and to make a comparison between this process and an existing wastewater treatment plant.

CHAPTER 2

LITERATURE REVIEW

2.1 Nitrogen in Wastewater

2.1.1 Introduction

Nitrogen in wastewater may present as organic nitrogen (such as amino acids, nucleic acid) and inorganic nitrogen (such as ammonium, nitrite and nitrate ions). Hydrolysis of organic nitrogen yields ammonia (equation 2-1 and 2-2), which may then be nitrified to nitrite and or nitrate.

$$\underset{\text{(Protein)}}{C-NH_2} + H_2O \xrightarrow{\text{Protease}} \underset{\text{(Organic C)}}{C-OH} + \underset{\text{(Ammonia)}}{NH_3} \qquad (2.1)$$

$$\underset{\text{(Urea)}}{H_2NCONH_2} + H_2O \xrightarrow{\text{Urease}} \underset{\text{(Bicarbonate)}}{HCO_3^-} + \underset{\text{(Ammonia)}}{NH_3} \qquad (2.2)$$

Nitrite and nitrate may be further converted into gaseous nitrogen by denitrifiers in the presence of organic carbon. In assimilatory nitrite or nitrate reduction, nitrite and nitrate are used as a nitrogen source for growth (for anabolism requirement). As a result, the reduced compounds are then converted into cell material. In dissimilatory reduction, the nitrite and nitrate are used as electron acceptors in energy generation (catabolism requirement). As a result, the reduced products are excreted into the environment. Madigan et al. mentioned that denitrification is one of these dissimilatory reduction metabolisms [3]. Wiesman presented a similar idea, he mentioned that nitrate and nitrite

can be reduced in nature anabolically and catabolically, but only the catabolic reduction of nitrate and nitrite are called denitrification [4]. Moreover, Henze et al. mentioned that denitrifiers are considered able to assimilate nitrate/ nitrite together with or without ammonium into biomass production (anabolic) [5]. Gottschalk mentioned that ammonia is a preferred source for nitrogen, which can be utilized practically by microorganisms. Nitrate and nitrite also can be used, but not all of microorganisms have this ability. Before they can be converted into organic substances, they have to be reduced to ammonia. Some bacteria have capability to fix nitrogen and then reduce it into ammonia [6].

In the current research the dissimilatory of nitrogen (i.e. catabolism reaction or denitrification) is considered using a pilot scale SBR.

2.1.2 Nitrogen Removal

Nitrogen removal is considered as one of the crucial issues in wastewater treatment, and it took a lot of concern by engineers and scientists in the last decades. The main reasons behind nitrification is to avoid the effect of ammonia on DO depletion in the receiving waters, to avoid “eutrophication”, and controlling of nitrogen for reuse purposes in groundwater recharge [1].

2.1.3 Pre-anoxic & Post-anoxic Denitrification

There are two main types of biological denitrification, pre-anoxic and post-anoxic denitrification [1]. The former, in which an anoxic tank followed by aeration tank and the nitrate which produced in the aeration tank is recycled to the anoxic tank and the electron donor is provided by the organic substrate. The latter, in which the aeration tanks

followed by anoxic tank, and the electron donor is provided by the endogenous decay because the organic substrate is consumed in the nitrification process in the aeration tank.

In 2006 K. Hamada et al. studied the deference between post and pre-denitrifications regarding nutrient removal efficiency. They conducted their experiments using two processes, the first is the modified UCT (University of Cape Town) as pre-denitrification process, and DEPHANOX as post-denitrification process. It was found that post-denitrification process have higher nutrient removal efficiencies than pre-denitrification process [7].

2.1.4 The Effect of pH on Nitrogen Removal Process

The importance of the Hydrogen-ion concentration expressed as pH comes from its effect on nitrogen removal (i.e. nitrification and denitrification processes) and on the growth rates of bacteria. For an optimum nitrification process, pH should be on the alkaline side. However, alkalinity is produced due to denitrification process. [1]. The nitrifying bacteria (i.e. Nitrosomonas and Nitrobacter) are very sensitive to pH change. For instance, the approximate optimal pH range for Nitrosomonas is (7-8), while for Nitrobacter is (7.5-8) [8].

A. C. Anthonisen mentioned in his paper (Inhibition of Nitrification by Ammonia and Nitrous Acid) in 1976 that pH can affects the rate of nitrification since the ammonia-ammonium and nitrite-nitrous acid equilibria depend on pH. And because both NH_3 and HNO2 inhibit nitrifying bacteria. If the pH of the extracellular surroundings is higher than the intracellular pH of a nitrifying bacteria, NH_3will breach the cell membrane and NH_4^+ will remain outside the cell. Likewise, if the extracellular surroundings pH is lower

than the intracellular pH of a nitrifying bacteria, HNO_2 will breach the cell membrane and nitrite will remain on the exterior [9].

Bhavender et al. mentioned in their paper (Nitrification and Nitrogen Removal) in 1977 that for an optimum nitrification, pH should be slightly on the alkaline side [10].

2.2 Sequencing Batch Reactors (SBR)

2.2.1 Introduction

The sequencing batch reactor (SBR) is an enhanced form of the activated sludge system in which the wastewater is treated in a fill and draw process. It has been used since 1920s [11] .It is considered as one of the integrated anaerobic-aerobic bioreactors .The elimination of undesirable materials is done by adding the wastewater to a separate batch reactor. The process of SBR contains five steps (filling, reacting, settling, decanting and idle). In the filling period, the wastewater and the substrate are added to the tank, and from 75% up to 100% of the volume could be occupied by the liquid in this phase. Using either mixing with aeration or mixing only is alternative in the filling period. The second phase is the react phase, during this phase and considering specific environmental situations; the substrate is removed and eliminated by specific bacteria. Thirdly, the settling period which comprises separation of the solids from the water allowing the treated water to be discharged. The decanting phase is the fourth phase of the process, in which the treated flow is removed. Lastly, the idle phase in which sludge is wasted, and it's only necessary in the multi-chamber systems [1].

The use of this process is apparent in the industrial wastewater treatment, because of its compactness and the high efficiency in COD, BOD, ammonia, and nitrogen removal.

However the use of SBR process in domestic wastewater treatment is rare, since domestic wastewater usually needs very large plants to be treated because of the high amounts of flows [12].

The conventional wastewater treatment plants have some disadvantages, though they can work properly under good installation and maintenance [13]. The SBR process as an enhanced form; presents flexibility in the treatment of inconstant influents, lowest operator interaction, alternative for aerobic or anaerobic environments in the same chamber, excellent oxygen interaction with microorganisms and substrate, lesser ground area, superior removal efficiency and requires less energy input. These benefits validate the latest rise in the operation of this method in the treatment of municipal and industrial wastewater. Nevertheless, inappropriate control of anaerobic and aerobic residences times in SBR system will bring about problems controlling the anaerobic–aerobic microbial groups and therefore the selection and enhancement of the required number of bacteria will be hard to accomplish [2].

2.3 Current and Previous Work on SBR

2.3.1 Nitrogen Removal from Municipal Wastewater

In our SBR system, and considering that only one tank is used to complete the nitrification and denitrification processes, mimicking the pre & post denitrification process was by controlling the time of filling (Tf) and the time of settling (Ts). As we mentioned before that the pre-anoxic denitrification consists of anoxic zone followed by aerobic zone; the pre-anoxic denitrification in the SBR system is by increasing the filling time without any aeration (anoxic zone) and decreasing the settling time. And for the post

anoxic in which the aerobic zone is before the anoxic zone; the settling time is increased (without any aeration) and the filling time was decreased (regardless wither the aeration takes place or not).

The following is a review to what has been discussed in the literature on nitrogen removal using SBR technique.

In 2016 a group of researchers lead by Zhang studied the effect of applying different aeration rates on the removal of nitrogen from municipal wastewater. They carried out their work on a lab scale SBR with active volume of 5l. The experimental work lasted for nearly 5 months to study the pre-anoxic and post-anoxic procedures of nitrogen removal. They found out that the COD removal was not affected by the aeration rates when the aeration rate is between 15-40 l/h. And for the same circumstances the TN increased and the decreased in the multiple post-denitrification processes [14].

In 2016 Ren-Jian Deng et al. conducted a research to study the performance of post-denitrification with deferent aeration rates on the removal of nitrogen using a lab batch SBR with an active volume of 5 L. Their results indicated that the removal performances of nitrogen and phosphorus both increased first and then decreased with increasing aeration rates and they also found that the aeration rate should be controlled at 25 L/h (dissolved oxygen = 2.7 mg/l) in order to achieve ideal removal efficiencies of total nitrogen (TN) [14].

In 2014 Zahra Bagheri et al. conducted a research on modeled pilot plant sequencing batch reactor using municipal wastewater from Ekbatan wastewater treatment in Tehran to study the influence of filling time, reaction time, aeration intensity as well as solids retention time (SRT) an mixed liquor volatile suspended solids (MLVSS) on the

characteristics of the effluent concentrations of COD and $NH_4^+ - N$. His pilot plant SBR comprised of plastic feeding tank of 300l volume, two reaction tanks of 340l each, two air compressors of 0.45 m^3/min and 0.88 m^3/min respectively, and settling tank of 550l. The study found that the appropriate removal efficiencies attained when the filling time was 1h, the reaction time was 6h, the aeration intensity was 0.88 m^3/min and the solids retention time was 30 days. High removal efficiency for COD and $NH_4^+ - N$ where attained 94% and 93% respectively [15].

In 2014 Zhiqiang Hu et al. have conducted an experiment to study the effect of hydraulic retention time in the removal efficiency of nutrients. Considering that the conventional treatment plants design and optimization need careful selection of the volumes of the aerobic, anoxic and anaerobic tanks as well as return activated sludge and mixed liquor recirculation, the SBR was a good choice for the researcher to conduct his experiment, since it doesn't include these problems. The researcher also used the MBR and he coupled it with the SBR to provide an MSBR system. The reason that the researcher used the MBR process is because of the higher flows and volumetric loadings that the MBR can provide. Hu found that the average concentrations of TKN in the effluent was obviously decreased in lower (HRT), for instance the TKN concentration of the effluent in 24 h. was 13 mg/l, but in 12 h. it was only 8 mg/l which is nearly 49% decrease and it decreased up to 7.8 mg/l in 6 h. In the other hand the nitrogen removal efficiency was increased significantly in shorter periods of (HRT) from 68% (in 24 h.) to 80% (in 12 h.) but it did not increase considerably after this time. This research evidently showed the huge effect of hydraulic retention time (HRT) on the removal of nutrients in MSBR [16].

In 2012 Jun Li et al. proposed a modified SBR (MSBR) to overcome the drawbacks of CASS and CAST processes. Their study investigated the performance of the MSBR, the $NH_4^+ - N$ & TN interaction in the removal of these constituents, and finally the interaction between anaerobic time and TN elimination. The research studied the anaerobic time influence on biological nitrogen removal using an improved sequencing batch reactor. They considered 1h anaerobic time and 0.5h aerobic time. Their SBR contains four champers of 300 m^3 with different anaerobic/ anoxic/ aerobic utilities. They considered 14h HRT, 15^oC controlled temperature, 3000 mg as MLSS and 28m^3/h as the rate of air flow from the compressor. Degradation of organic substrate as well as nitrification occurred on the 4 tanks serially. In order to avoid the effect of high organic loading; the permanent microorganisms grew in different champers. They ran the system for one year to study the variations of $NH_4^+ - N$, total phosphorous, total nitrogen, and COD. The ammonia and total nitrogen removal efficiency were 98 and 52 as average. They found that the TN removal efficiency was influenced by the influent of ammonia-nitrogen concentration [17].

J. Keller et al. in 2007 noticed that the aerobic duration is strongly affected the nitritation (removal of nitrogen via nitrite) when he and his group were searching the latter process. The study found that in the process of removing the nitrogen via nitrate, the SBR removed 95% of the ammonia in the wastewater by oxidation and 90% removal of nitrogen. Also it found that the percent of nitrogen removal is corresponding to the percent of COD/TKN. In his model the study indicated that nitritation is strongly affected by the ammonium oxidizing bacteria (AOB) and nitrite oxidizing bacteria (NOBs) growth rates. It was said in the study that in order to maintain successful nitritation,

AOBs growth rates must be larger than NOBs growth rates. And this is very important for nitrite accumulation. It was also stated that Aerobic time control seems fairly efficient in achieving nitritation for this system but its use is perhaps inadequate for batch or plug flow methods with an obviously visible difference in the ammonium concentration profile. Other ammonium oxidation patterns with low ammonium concentration at all times (such as an oxidation ditch) may not permit a satisfactorily huge variance in the AOB and NOB growth rates due to the impact of the high ammonium Ks values relative to nitrite Ks values [18].

J. Y. Hu and a team of researchers established a study in 2005 to investigate the effect of three factors (batch loading, feed pattern, and mixing/aeration ratio) on the removal of nitrogen from municipal wastewater. They used a cylindrical container with a volume of 12l as a sequencing batch reactor to carry out the experiments. They used the Tagushi method in the experimental design to reduce the number of experiments ended up with 9 runs for each scenario of the experimental work. The scenarios consisted of studying each factor alone, and combinations between them which were (batch loading and feed pattern, batch loading and mixing/aeration ratio, feed pattern and mixing/aeration ratio). They found out that the optimum way for the suspended growth to perform perfectly is by using batch loading rate of 0.17 mgBOD/mgMLVSS.d, and initial feed pattern, and 1/1 as (mixing/aeration ratio). The result according to using this method were found to be Suspended solids removal (98.5%), TN (93.8), TP (98.2), and BOD_5 (99.6) [19].

2.3.2 Carbon to Nitrogen Ratio (C/N)

The particular nutrients content in wastewater have to meet the demands of microorganisms in the activated sludge, hence balanced relationship between N and C should be existed. This is critical to the efficiency of the biodegradation processes. In the course of aerobic wastewater treatment, the C/N ratio must be in the range between 100/10 and 100/5. Though, all types of industrial plants, regional variations in eating lifestyles (throwing away different kitchen wastes in the sewers), and the nature of the soil and drinking water cause wastewater to change broadly in its composition. Practice has proven that the C/N ratio is about 100/20 in municipal wastewater. It is easy to remove the additional nitrogen compounds by contemporary techniques. Furthermore, a specific amount of readily biodegradable carbon should be available in order to achieve efficient denitrification. The wastewater will pass the settling tank with a BOD_5/N ratio of 100/25, but if the ratio dropped beneath 100/40, the denitrification process will be affected and nitrate will present in the effluent [20]. Two techniques should overcome this issue, the first is to bypass the primary treatment, and the second is raising the denitrification volume. But these techniques might fail to solve the problem, and in such a case external carbon source will be inevitable solution.

In nitrogen removal biologically, scarce of biodegradable organic substrate to nitrogen matters (i.e. low C/N ratio) is considered as one of the restrictive factors [21].

Along with other heterotrophs, denitrifying bacteria is recognized to vie for a carbon source, which means that a small carbon to nitrogen ratio in the influent accelerate the carbon deficiency, producing an unstable instantaneous nitrification and denitrification.

The average COD/TKN ratio in domestic wastewater was reported to be beneath 6 [22]. Beccari et al. validated that biological removal of nitrogen can be appropriately accomplished over a ratio of COD/TKN of 13 [23]. Q. Fontenot et al. proved that C/N of 10:1 created top results in terms of maximum nitrogen and carbon removal from the wastewater [24].

In 2008, Daekeun Kim et al. used a two stage SBR with independent nitrification to investigate the removal of COD, nitrogen, and phosphorus from a low carbon to nitrogen municipal wastewater (C/N ranged from 1.1 to 7.4). They used three lab scale tank reactors with 0.01125 m^3 each as illustrated in Fig 2-1. The process generally has eight stages, which are done in order starting with filling moving to contacting and then settling and decanting, nitrification, refilling, reacting (anoxic/oxic), and finally idle. Each of these steps is shown in Fig. 2-2.

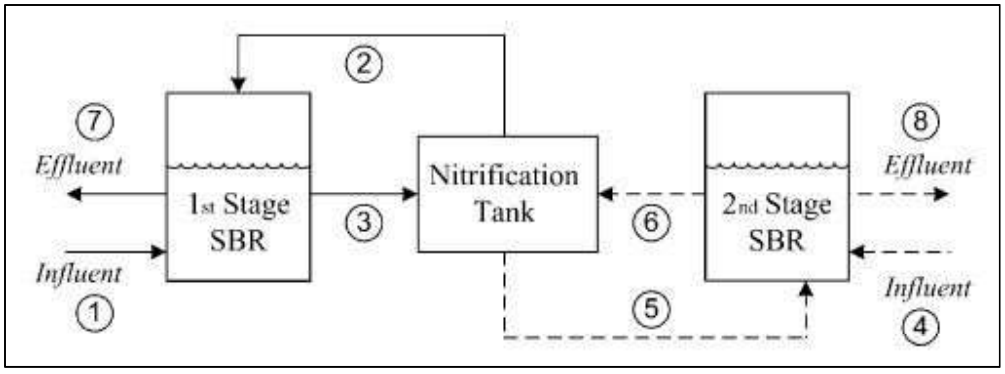

Figure 2-1 The improved two-phase application

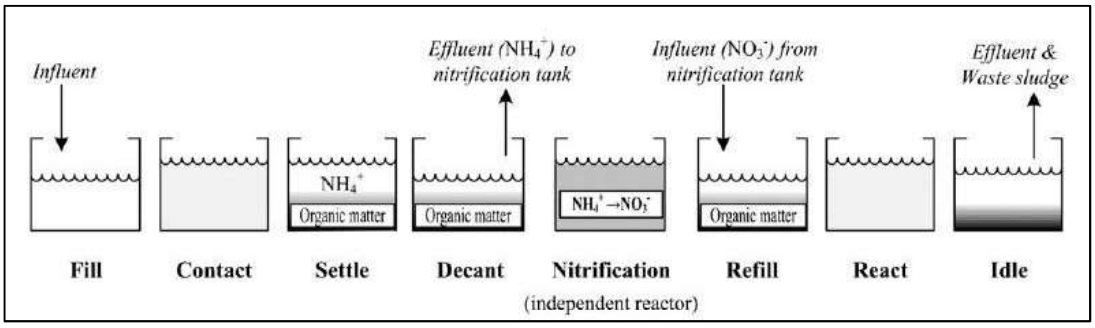

Figure 2-2 Process policies for the application of the improved two-phase SBR method.

They reported an 87% removal efficiency of COD, 81% removal efficiency for TN, and 60% removal efficiency for TP. The fraction of organic substrates and ammonia nitrogen from the influent has been established by the contact time. The ammonia nitrogen has then been nitrified as much as 70% in the external reactor. The system has achieved efficient total nitrogen, total phosphorus, as well as chemical oxygen demand lacking added carbon sources at all, [25].

In 2012, Yunxiao et.al investigated the capability of a laboratory scale SBR in the removal of nutrients such as nitrogen and phosphorus. Using different C/N ratios they found out an optimum C/N for TN removal which is 10:1 [26].

2.3.3 Dealing with Low C/N Wastewater

Yongzhen et al. established a study on July 2016 to investigate the effect of sludge fermentation on the nitrogen removal efficiency from low C/N wastewater using a 9l sequencing batch reactor. By controlling the pH and DO they found that the removal efficiency of TN reached 93.5% [27].

2.3.4 Chemical Oxygen Demand (COD)

The Chemical Oxygen Demand in wastewater is defined as the equivalent oxygen of the organic matter which can be chemically oxidized using dichromate in an acid solution [1]. The SBR is known for its capacity to remove high amounts of COD [28].

The SBR is well known as we mentioned before in the treatment of industrial wastewater. However the use of SBR in municipal wastewater had some focus in the literature, not to mention in COD removal.

In 2009 at Zhuzhuanling Wastewater treatment plant, eastern Hefei, China, Bing-Ni et al. conducted a study using a pilot-scale SBR with 1m3 volume, 0.5 m internal diameter to treat a low strength municipal wastewater in which COD was less than 200 mg/l. The DO of the water was kept at 2.0mg/l, HRT at 6-8 h, and pH at 6.5-7.5. And they achieved 90% COD removal efficiency [29].

In 2003, in Izmir, Turkey Uygur and Kargi established a research on synthetic wastewater with COD = 1200mg/l and COD/N/P = 100/5/1.5, to study the effect of salt on the inhibition of biological nutrient removal. They use a four steps SBR (anaerobic/ oxic/ anoxic/ oxic). Varying with salt content from 0% to 6%, they came up with this formula for the removal of COD ($r^2 = 0.97$):

$$ECOD = 93.42 - 9.83\ (T)$$

Where ECOD is the COD removal percentage, and (T) is the salt concentration percentage [30].

2.3.5 The Need for Decentralized Wastewater Management Systems

Nowadays in excess of 2.5 billion people suffer from quality water deficiency [31]. The population growth and the high costs for centralized treatment plants lead to the necessity of new technologies. Decentralized wastewater management systems are considered by many researchers as an alternative to the normal centralized system. Decentralized Wastewater Management DWM may be defined as the collection, treatment, and removal/recycle of wastewater from separate homes, clusters of homes, isolated groups at or nearby the point of waste production [32].

The SBR is not well known as a decentralized wastewater treatment process. Though, a study was conducted by P. Melidis in 2008 presented the operational data of a small Decentralized wastewater system to estimate the SBR performance. The wastewater treatment plant was sited in a four people home in Greece, it involved in two settling tanks of total volume 3.25 m^3 (i.e. 1.62 m^3 each). The influent BOD_5 and COD concentrations were 270 mg/l and 463 mg/l. The BOD_5 and COD were measured and associated with the influent. He found that the removal efficiencies were 90.9% for BOD and 86.9% for COD (mean values) [33].

2.3.6 The Scale of Use

The SBR process is adopted in industrial wastewater treatment plants as the scale is smaller than in municipal wastewater treatment plants. But very few work was done in the literature considering larger scale processes.

In Central and Eastern Europe, new waste water treatment technologies have been adopted and installed. The biggest one is based on (SBR) process built in Nowy Targ,

Poland. This plant showed that every new treatment plant should combined with the system of water and wastewater already existed. Biological phosphorus removal as well as nitrogen removal have been achieved in the effluent with minimum concentrations (1 mg/l for phosphorus removal) [34].

2.4 Wastewater Treatment Plant Assessment

The evaluation of the removal efficiency of wastewater treatment plants (WWTPs) is considered as one of the major assessment factors of WWTPs along with other aims of wastewater treatment plants assessment such as operational cost assessment, and the impact of the effluents on the environment [35].

In 2012 Ozkan established a study aiming to make an assessment of a large scale wastewater treatment plant in turkey namely (Kayseri wastewater treatment plant). The KWWTP served an 800,000 population with average flow of 110,000 m^3/d. The assessment consisted of collecting and analyzing four years of data from 2006 up to 2009. The data combined of Influent and effluent COD, BOD, SS, TN, and TP. The assessment of the wastewater treatment plant consisted of calculating the removal efficiencies of the above parameters. He revealed that the minimum COD removal efficiency during the four years was 94%, for BOD was 98%, for SS was 94%, for TN was 82%, and for TP was 71%. And all the characterizations of effluent did not exceed the limits [36].

In 2004 Hospido et al. conducted a research on the performance of a municipal wastewater treatment plant considering eutrophication. They studied the impacts on the environment corresponding to nitrogen removal by nitrification and denitrification

processes. Their research established empirical data among two years (2000 and 2001) measuring the nitrogen on monthly basis. They made a phisico-chemical assessment to find that nitrogen removal has excellently reduced the environmental impact of the discharged water [37].

CHAPTER 3

RESEARCH OBJECTIVES

The Kingdom of Saudi Arabia is facing lots of obstacles regarding water deficiency. The Annual Performance Report of the Ministry of Environment Water and Agriculture mentioned that the consumption of drinking water in 2016 reached 3129 Million m^3, the Eastern Province share was 659 Million m^3 [38]. It was reported in the same report that the treated wastewater in Eastern Province was 524.6 Million m^3 and only 16% of it was reused [38]. Al-Jasser in 2009 mentioned that most of the existing wastewater treatment plants do not meet the standards of Meteorology and Environmental Protection Agency (MEPA) [39]. Because of these challenges, KSA needs to properly utilize its water resources including treated wastewater. Therefore, there is a continuous need to investigate technologies that would be more efficient than current technologies in treating wastewater and meeting the reuse needs.

In the current research, the main aim is to consider SBR process and a selective community namely North Sewage Treatment Plant (NSTP) to achieve the following specific objectives:

1. Assess the quality of water at North Sewage Treatment Plant, NSTP.
2. Investigate the biological nitrogen removal efficiency at laboratory pilot scale SBR and study different scenarios (pre-anoxic denitrification and post-anoxic denitrification) to select the optimum conditions.
3. Investigate the removal efficiency of COD, Turbidity and TSS at the SBR.

4. Explore the possibility of using SBR technology as a decentralized wastewater treatment plant for treatment and reuse in the KSA.

CHAPTER 4

MATERIALS & METHODS

4.1 Overview

A pilot scale plant SBR was used to treat raw domestic wastewater used as the influent was obtained from local wastewater treatment plant namely Doha NSTP. The purpose was to assess and to make a comparison between the conventional wastewater treatment plant and the SBR as a technique of treating and removal of nitrogen from municipal wastewater, and to investigate whether SBR process can be used as decentralized wastewater systems or not. The SBR design procedure for BOD removal and nitrification as well as nitrate removal (i.e. denitrification) are considered as a reference for the design in this research and they were adopted from Metcalf and Eddy [1]. In addition to that, the characterizations of the influent and effluent wastewater were according to the standard methods for examination of water and wastewater [40], and HACH [41].

4.2 Experimental Setup

At the beginning of the work a fully automated 16*l* SBR was chosen to carry out the experiments. But during the work the apparatus was damaged. Seeking for a good maintenance was a very hard task and it took us months. At the end of the day we decided to change the apparatus to a 250*l* SBR which was bigger than the first one. Maintaining a good experimental setup and automation to the new SBR was a hard but a great experience to us.

The experimental setup of the pilot plant SBR consisted of two phases:

The 1st phase was insuring the capability of the SBR in removing the nutrients from the wastewater. This phase consisted of (one week, three batches per day, and different filling, aeration, settling, and decanting times). The operating the SBR was using wastewater taken from North sludge treatment plant (NSTP) (see appendix A).

The 2nd phase was automating the system. Furthermore, in this process the ARDUINO program language was used to write a code. Using a microcontroller was the best choice to automate the SBR filling, aeration, settling, and decanting times (see Appendix D).

Two scenarios were investigated, pre-denitrification and post-denitrification processes in order to find the most appropriate process of nitrogen removal. Each scenario consisted of 3 samples, and different batches with different time of filling and time of settling. The time of aeration was in the range of 3-6 hours for operational issues.

4.3 Equipment of the Pilot Plant SBR

4.3.1 SBR Components

Fig 4-1 shows a Schematic diagram of the pilot scale SBR. And its components are as follows:

1. Oxidation reactor, R1.
2. Feeding tank, D1.
3. Static settler, D2.
4. Feeding gear pump, G1.

5. Air compressor, P1.
6. Air diffuser for reactor, R1.
7. pH - meter, AI1.
8. Oximeter, AI2.
9. Temperature indicator, AI3.

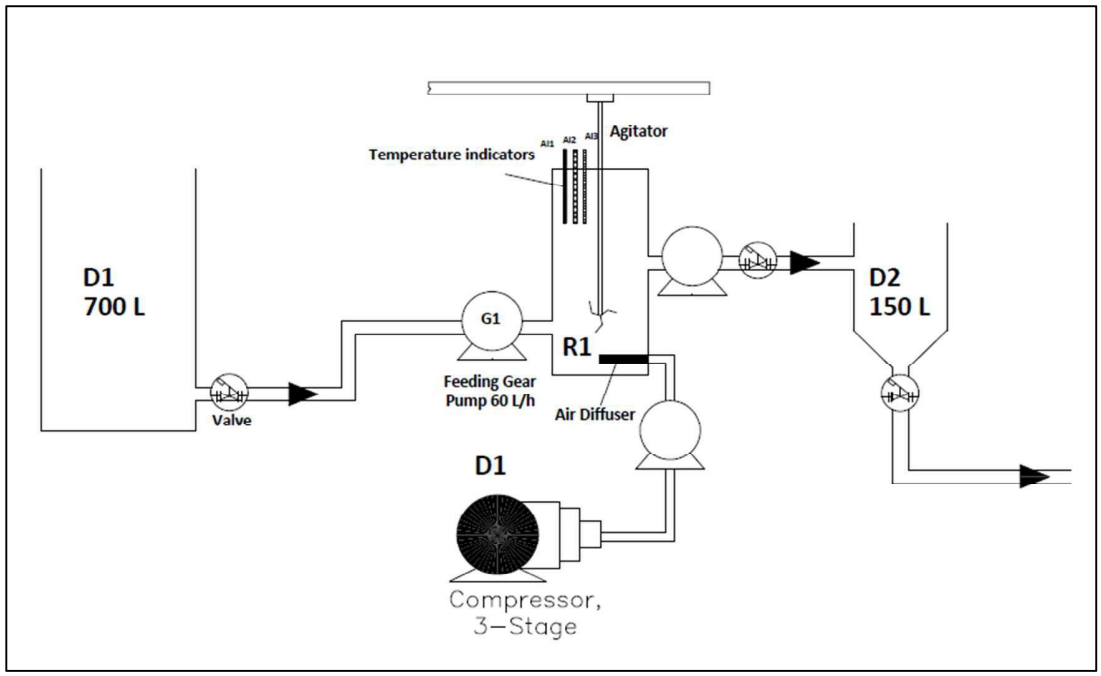

Figure 4-1 Schematic diagram of the pilot scale SBR

4.4 SBR Treatment Process Description

4.4.1 Samples Collection

The untreated wastewater samples were collected from local wastewater treatment plant NSTP from the influent and secondary treatment tank in 1*l* bottles weekly for 4 months to assess the efficiency of the existing plant. The main samples used for the nitrogen removal study were collected in 250*l* containers from the influent as a raw wastewater to the SBR pilot plant system.

4.4.2 Wastewater Characterization

The influent and effluent wastewater for the local wastewater treatment plant were characterized for the following parameters:

pH, Turbidity, nutrients (Nitrogen- ammonia, nitrite, nitrate, Kjeldhal nitrogen), organic load (including total COD, dissolved COD and BOD). The Standard Methods for examination of water and wastewater [40] as well as HACH method of characterization, were used as a manual for characterization of wastewater.

pH: pH was measured using a pH meter after calibration at each use.

Turbidity: Nephelometric method was followed to measure the turbidity of the wastewater and treated water using nephlometer.

Total solids: TS was calculated by drying the well mixed wastewater sample at 103℃ and weighing it and subtracting the weight of the dish after igniting it at 550℃.

Total nitrogen: TN was calculated using HACH method.

Nitrate: Nitrate was calculated using HACH method.

Nitrite: Nitrite was calculated using HACH method.

Ammonia: Ammonia was calculated using HACH method.

TKN: By following the standard method [42] of converting all the nitrogen to ammonia and then digestion till all the fumes disappear. And then measuring the samples after completing the volume to 100ml.

BOD: Using the automated method of measuring BOD. Using 164 ml Oxi-Top vessel, BOD bottles, funnel, sterilizers, and automatic BOD reader.

COD: Using HACH method.

sCOD: Using HACH method after filtering the samples using 0.45µm filter papers.

4.5 Assessment of NSTP Wastewater Treatment Plant

The NSTP as shown in Fig 4-2, is located in the Dhahran area. It is an activated sludge wastewater plant, with an average of 52,996 m^3/d. Flow enters the plant via gravity sewers from three areas:

1- Doha home ownership areas
2- Saudi Aramco Dhahran areas
3- King Fahd University of Petroleum and Minerals

The headworks area consists of mechanically cleaned step screens followed by vortex grit removal chambers. Following the headworks, the screened and de-gritted flow is distributed to the aeration tanks where biological stabilization of organic matter is accomplished.

The aerated effluent exiting the aeration tanks is conveyed to secondary settling tanks for removal of solids.

Figure 4-2 North Sewage Treatment Plant- Dhahran

CHAPTER 5

RESULTS AND DISCUSSION

5.1 Assessment of North Sewage Treatment Plant NSTP

Improving the discharge quality and avoiding eutrophication in the receiving body requires removal of pollutants in wastewater treatment plants such as North Sewage Treatment Plant (NSTP). The role of the secondary treatment is to remove the organic substances and the nutrients from the wastewater. A complete table of the analyzed influent and effluent samples for pH, conductivity, Turbidity, Total suspended solids, Ammonia, DCOD, BOD, DO, Nitrite, and Phosphorus, during the four months is given in appendix A, and a summarized one is shown in Table 5-1 shows also the maximum allowable limits for the influent and secondary treated wastewater. In this discussion part we considered the influent and effluent characterizations of NSTP as the main elements of assessment and we compared the effluents to the regulations of Ministry of Environment, Water and Agriculture in Saudi Arabia (MEWA). No heavy metals concentration problems were observed.

Table 5-1: Statistics of North Sewage Treatment Plant NSTP

Number	Parameter	Average influent	Average Effluent	Influent Standard Deviation	Effluent Standard Deviation	Min Influent	Max Influent	Min Effluent	Max Effluent	Allowable Influent limits (MEWA)	Allowable Effluent limits (MEWA)
1	pH	7.44	7.61	0.24	0.25	7.13	8.06	7.12	8.23	0.00	8.40
2	Conductivity ms/cm	4.51	6.51	0.75	7.50	2.26	5.23	3.94	35.50	-	-
3	Turbidity NTU	81.31	2.63	37.81	1.20	34.60	187.00	1.20	5.11	-	5.00
4	TSS mg/l	243.58	6.21	84.56	3.13	84.90	381.60	2.87	12.26	600.00	40.00
5	Ammonia mg/l	18.71	0.22	6.21	0.38	7.03	35.00	0.00	1.58	80.00	5.00
6	Nitrate mg/l	0.73	3.46	0.60	1.20	0.14	2.01	0.96	6.07	-	10.00
7	Nitrite mg/l	0.05	0.04	0.03	0.02	0.00	0.12	0.00	0.10	-	-
8	DCOD mg/l	69.36	46.47	18.73	66.27	45.40	128.00	13.80	300.00	-	-
9	TCOD mg/l	179.57	64.30	70.85	78.42	68.00	359.00	15.40	357.00	-	-
10	BOD mg/	79.88	8.63	25.79	5.18	48.00	144.00	2.00	18.00	500.00	40.00
11	DO initial mg/l	4.33	4.12	2.14	1.48	0.77	8.70	1.00	6.50	-	-
12	Phosphorus mg/l	2.23	1.21	1.26	1.14	0.30	4.54	0.00	4.38	-	-

5.1.1 Nitrogen Removal

5.1.1.1 Ammonia-Nitrogen ($NH_4^+ - N$) Removal

Ammonia-Nitrogen is the part of nitrogenous compounds which consumed in the nitrification zone. Fig 5-1 shows the weekly variation of influent, effluent, and removal efficiency of ammonia-nitrogen in NSTP over four months. The fluctuation of ammonia influent concentrations is within the range of normal oscillation of typical municipal wastewater ammonia-nitrogen concentrations (10-25 mg/l) except for the 7th week (7mg/l) and 16th week (35mg/l).

Ammonia-nitrogen is converted to Nitrite by Nitrosomonas and then to Nitrate by Nitrobacter. As it was mentioned in the paper (Nitrification) [8] which published by the Environmental Protection Agency EPA that the nitrifyincg bacteria (i.e. Nitrosomonas and Nitrobacter) are very sensitive to pH change. For instance, the approximate optimal pH range for Nitrosomonas is (7-8), while for Nitrobacter is (7.5-8). However, the pH change of NSTP influent was between 7.13 and 8 as shown in Fig 5-4 in the pH discussion part.

Ammonia-nitrogen concentrations in the effluent were minimal and less than 1 mg/l which is a typical secondary nitrified effluent. Furthermore, ammonia-nitrogen concentrations were found to be less than the allowable maximum values (5.0 mg/l) according to the Ministry of Environment, Water and Agriculture [43], proving that the process is very successful in the removing of Ammonia with 98.8% over all removal efficiency as shown in Fig 5-1.

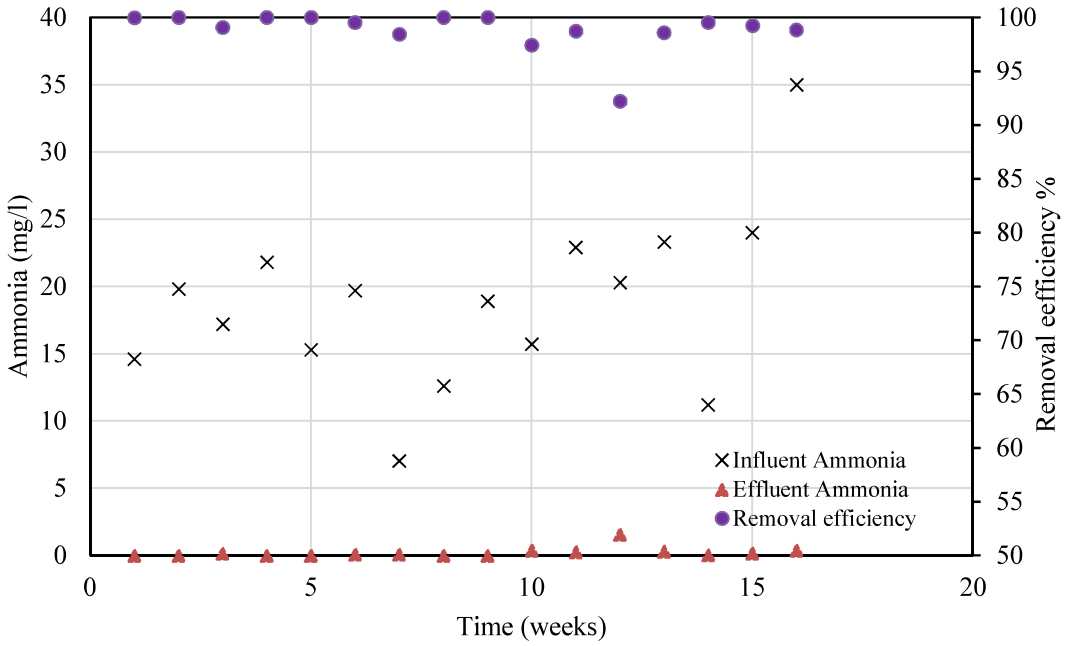

Figure 5-1 The Variation of Ammonia in the Influent and Effluent of NSTP

5.1.1.2 Nitrite (NO_2^-) and Nitrate (NO_3^-) Removal

Fig 5-2 shows the weekly variation of nitrite influent and effluent concentrations of NSTP. Nitrite in wastewater is considered as an indicator of toxicity, and its concentration is minimal in municipal wastewater. Human sewage, livestock manure, fertilizers as well as erosion of natural deposits are considered as sources of nitrite in wastewater. Typical municipal wastewater doesn't contain such sources and this is the reason behind the small concentrations in the influent.

Fig 5-3 illustrates the change of nitrate in the influent and effluent of NSTP. The influent nitrate concentrations were minimal also. Like nitrite, nitrate sources, in wastewater are human sewage, livestock manure, fertilizers and erosion of natural deposits. Based on the stoichiometry of biological nitrification, the ammonia is converted to nitrite by Nitroso-bacteria and then to nitrate by Nitro-bacteria by the following equations in which finally every mole of ammonia produces two moles of nitrate.

$$2NH_4^+ + 3O_2 \rightarrow 2NO_2^- + 4H^+ + 2H_2O \qquad (5.1)$$

$$2NO_2^- + O_2 \rightarrow 2NO_3^- \qquad (5.2)$$

$$NH_4^+ + 2O_2 \rightarrow 2NO_3^- + 2H^+ + H_2O \qquad (5.3)$$

The values of effluents nitrate concentrations were greater than the influent concentrations, and that because of nitrification process, in which the ammonia is converted to nitrite and nitrate. But still it is lower than the maximum allowable level of pollution (10.0 mg/l) according to the regulations of the Ministry of Environment, Water and Agriculture [43].

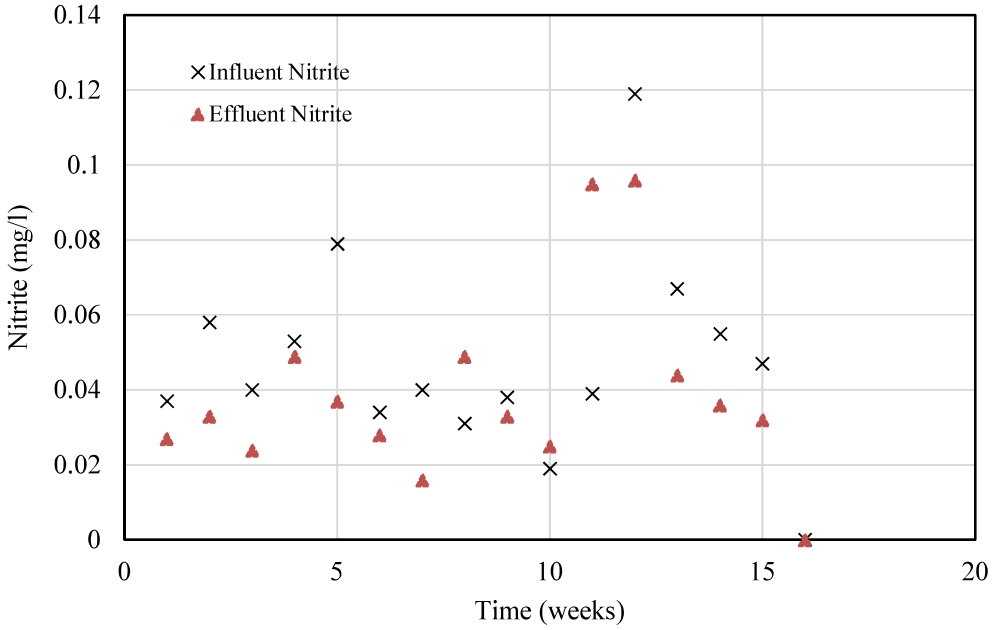

Figure 5-2 The Variation of Nitrite in the Influent and Effluent of NSTP

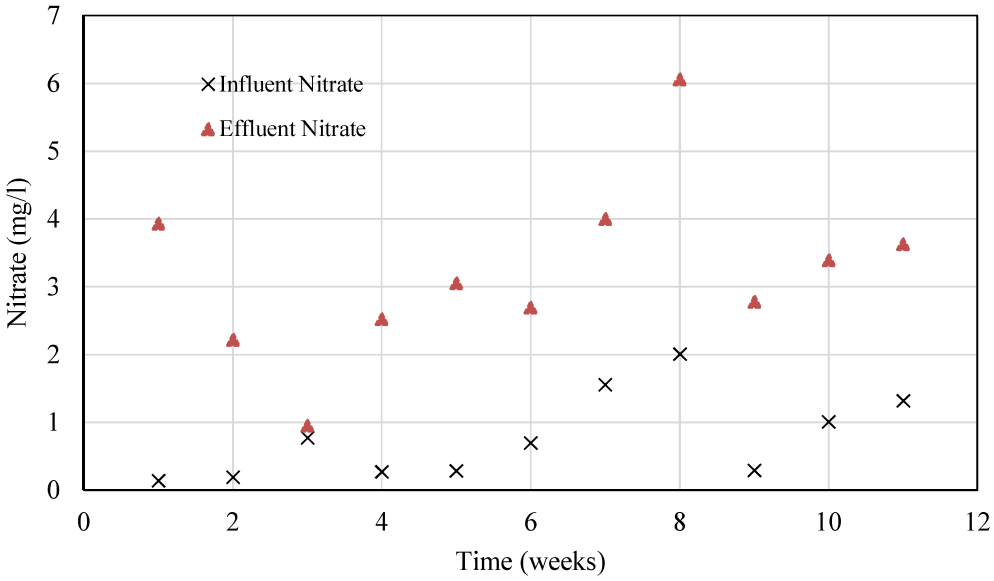

Figure 5-3 The Variation of Nitrate in the Influent and Effluent of NSTP

5.1.2 pH

As in Fig 5-4, all influent and effluent pH values fall within the range of 7 and 8.2. For nitrification to be occur, it is important to mention that the value of pH should not be lower than 7, which indicates that the values of pH in NSTP wastewater are suitable for nitrification to take place. De-nitrification process generally elevates the value of pH which is very clear in the Fig 5-6. For the points in which pH in the effluent are less than the influent, one can say that denitrification did not take place which is very clear at the third week in Fig 5-2 as the Nitrate concentration in the influent and effluent were the same. It is important also to mention that pH values of the influent and effluent lie within the MEWA regulations.

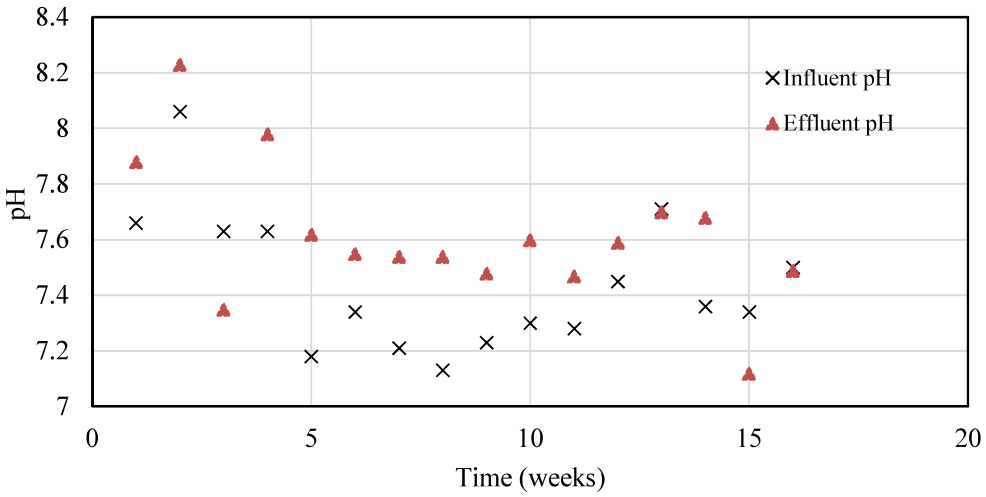

Figure 5-4 The Variation of pH in the Influent and Effluent of NSTP

5.1.3 Biochemical and Chemical Oxygen Demand (BOD and COD) Removal

Fig 5-5 shows the weekly variation of BOD influent and effluent concentrations as well as removal efficiency of NSTP. Influent BOD shows low concentrations. Low BOD concentrations means low carbon source which consequently will affect nitrogen removal. All effluent BOD concentrations do not exceeded 20 mg/l as shown in Fig 5-5, which indicates a good removal efficiency of the system. The oscillation of BOD in the influent was shown a bit high, and that might be a reflection to the BOD during the day, since the time of sampling was not always the same. It is very clear that such a treated water quality can strongly be recycled and can be used in irrigation. Tertiary treatment of course will reduce this concentrations even less.

Similar to BOD, the trend of the influent COD as shown in Fig 5-6, vary from week to week which is also attributed to time of taking the sample. The effluent concentrations vary around 50 mg/l.

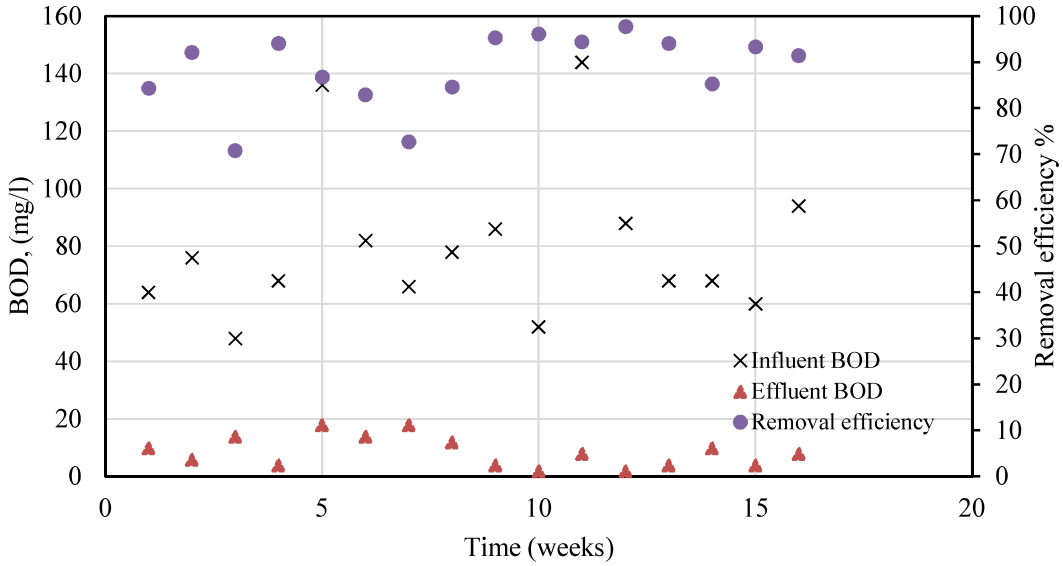

Figure 5-5 The Variation of BOD in the Influent and Effluent of NSTP

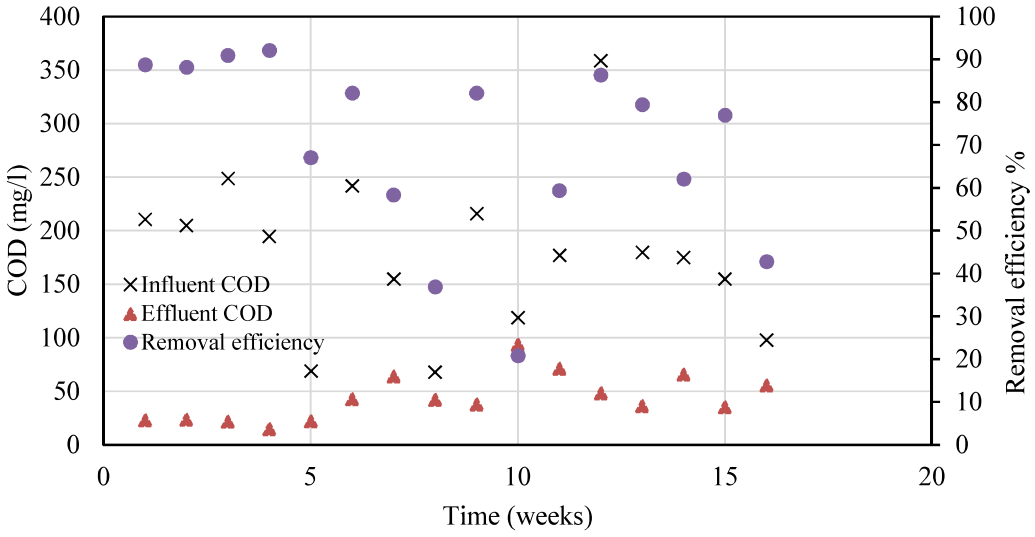

Figure 5-6 The Variation of COD in the Influent and Effluent of NSTP

5.1.4 Turbidity Removal

Significant removal efficiency of Turbidity at NSTP was achieved, since the effluent concentrations were less than 5.00 NTU except for the 13th week it was 5.11 NTU as shown in Fig 5-7, with average removal efficiency of 96.2% which is excellent.

5.1.5 Total Suspended Solids (TSS) Removal

Attainment of high removal efficiency for total suspended solids is a very crucial factor in water quality. Fig 5-8 shows that the maximum TSS in effluent is 380 mg/l which is less than 600 mg/l (according to MEWA), and the maximum TSS concentration in the effluent is 12.3 mg/l, which is less than 40 mg/l according to MEWA regulations, giving us a good picture regarding TSS removal in NSTP. The average removal efficiency is 96.7%.

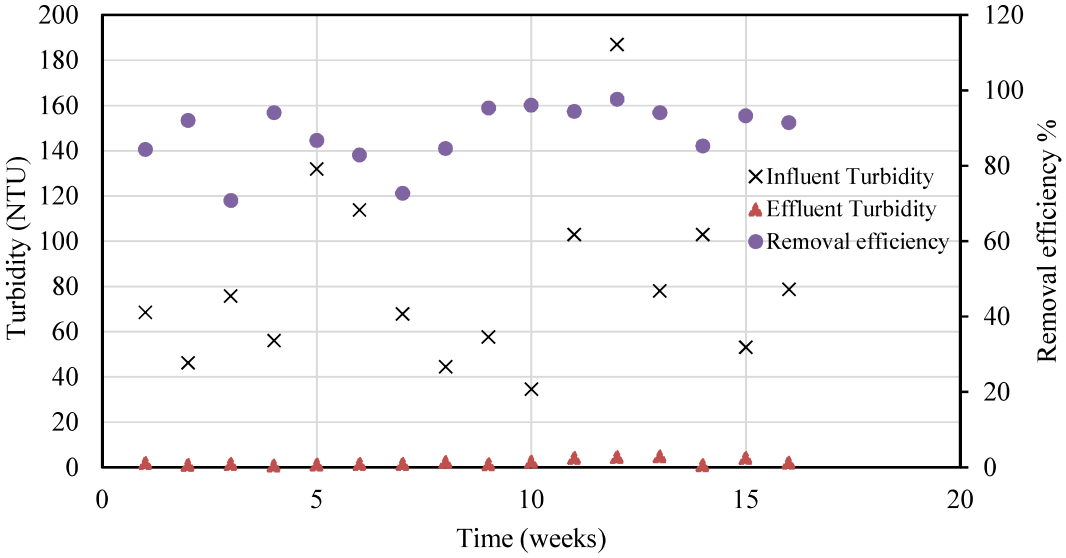

Figure 5-7 The Variation of Turbidity in the Influent and Effluent of NSTP

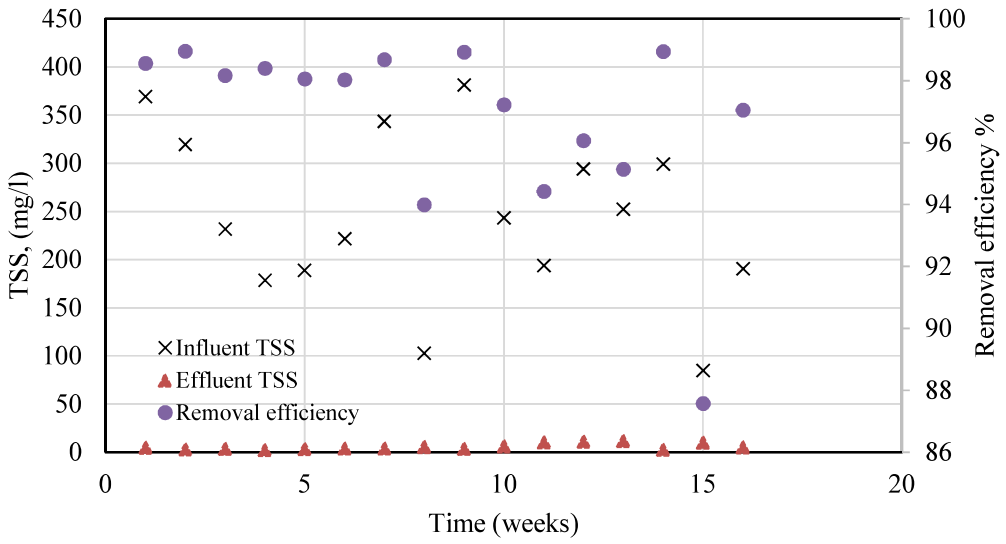

Figure 5-8 The Variation of Total Suspended Solids in the Influent and Effluent of NSTP

5.2 SBR Process

5.2.1 Experimental Setup (Challenges and Improvements)

At the beginning of the work a fully automated 16*l* volume SBR was chosen to carry out the experiments. But during the work the apparatus was damaged. Seeking for a good maintenance was a very hard task and it took us months. At the end of the day we decided to change the apparatus to a 250*l* volume SBR which was really bigger than the first one. Maintaining a good experimental setup and automation to the new SBR was a hard but a great experience.

The experimental setup of the pilot plant SBR consisted of three phases:

The first phase was designing an excel sheet contains all the necessary parameters .The design of SBR was adopted from Metcalf and Eddy [1], it consists of seventeen steps as shown in Appendix B. The influent data was collected and considered as input data for the design, and the outputs of the design were used in the experimental setup All desing result were organized in Appindex C.

The second phase was automating the system. And in this process the ARDUINO software was used to write a code for automation. Using a microcontroller was the best choice to automate the SBR filling, aeration, settling, and decanting times (see Appendix D).

The third phase was insuring the capability of the SBR in removing the nutrients from the wastewater. This phase consisted of (one week, three batches per day, and different filling, aeration, settling, and decanting times). The operation of the SBR was done using

wastewater which taken from North sludge treatment plant (NSTP), and significant removal efficiencies were achieved (see appendix E).

The SBR influent and effluent results are collected in Appendix F.

5.2.2 Low Organic Feeding

The BOD concentrations of the wastewater ranged between 70-150 mg/l, which were low in comparison to the typical BOD values in the wastewater 100-500 mg/l [44].

5.2.3 Total Nitrogen (TN)

5.2.3.1 TN Removal in Post- anoxic Denitrification Process

The post-denitrification process was presented here using the settling time as anoxic zone in which denitrification took place. Fig 5-9 illustrates the change in Total Nitrogen (TN) removal efficiency with respect to settling time (Ts) in the post-denitrification process. TN removal increased slightly from 26% to 30% when the settling time increased from 3 to 4 hours. By 6 hours, TN removal reached a plateau on 62% removal efficiency. So, one can say that 6 hours is the optimum settling time for TN removal.

These findings showed that post-anoxic denitrification can prefroms better than pre-anoxic denitrification. The post-anoxic denitrification using DEPHANOX process achieved higher removal efficiencies that pre-anoxic denitrification (K. Hamada et al. 2006) [7].

What actually happens here is that facultative heterotrophic bacteria uses the nitrate which is the remaining nitrogenous compounds from nitrification, as electron acceptor in the oxidation reduction reactions and then convert it to nitrogen gas It is clear that the TN removal efficiencies are low at lower values of Ts and high at higher values of Ts in a behavior of step function. In the nitrification phase, ammonia-nitrogen is converted into Nitrite by Nitrosomonos bacteria and then to Nitrate by Nitrobacter bacteria, and the nitrification which took place in aeration time (Ta) is followed by settling phase, the settling phase is an anoxic zone in which the remaining nitrogenous compounds fraction (i.e. NO_x) serve as electron acceptors which means they are converted into nitrogen gas. This process is completed by the facultative heterotrophic bacteria which uses the nitrate as electron acceptor in the oxidation reduction reactions. Since the settling time represent

the time of the anoxic zone, it does mean that any increase in this time will result in higher removal efficiency, which agrees with our results in Fig 5-9. An average removal efficiency of 50% and maximum removal efficiency of 64% were achieved in this process.

5.2.3.2 TN Removal in Pre- anoxic Denitrification Process

Pre-denitrification process was presented using the filling time in which denitrification took place. Fig 5-10 shows the change in TN removal with filling time in pre-denitrification process.

Pre anoxic denitrification is limited due to dissolved oxygen (DO) return from the aerobic zone, and dilution of influent carbon which eventually lead to incomplete NOx removal (Matt Winkler et al 2011).

Fig 5-11 shows the change in average TN removal with C/N ratio in pre-denitrification process. The average TN removal efficiency was 17% at C/N of 5.67, and increased to 20% at C/N of 6.53, and to 28% at C/N of 7.43. So, TN has an increasing trend with C/N ratio.

These findings showed that as C/N ratio increases, total nitrogen removal efficiency increases as will. And this agrees with the findings of Yunxiao Jin et al. (2012) as he found that the average removal efficiencies of total nitrogen (TN) reached 87.2%, 65.4%, 46.1% respectively as C/N ratio changing from 10:1, 5:1 to 3.3:1. Since, he used synthetic wastewater he was able to control C/N ratio, but as we deal with municipal wastewater our rule was to observe the effect of C/N ratio on the removal of nitrogen.

The reason why C/N ratio increases nitrogen removal is that as carbon to nitrogen ratio increases that means more electron donors to electron acceptors, which means more potential for the dissimilating processes to be occur. Moreover, just like other heterotrophs, denitrifying bacteria is recognized to compete for a carbon source, which means that higher C/N ratio in the influent deaccelerate the carbon deficiency, creating a stable nitrification and denitrification. An average removal efficiency of 21.7% was achieved in this process, and maximum removal efficiency of 30%.

And obviously we can recognize that the post-denitrification process is better than pre-denitrification process in terms of TN removal.

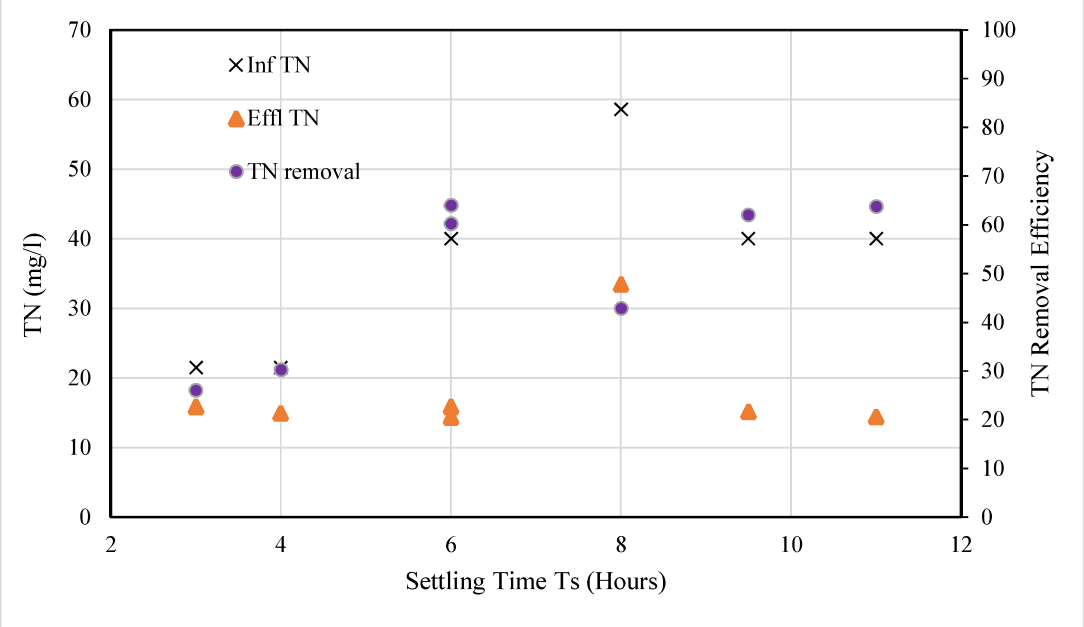

Figure 5-9 TN removal with settling time in post-denitrification process

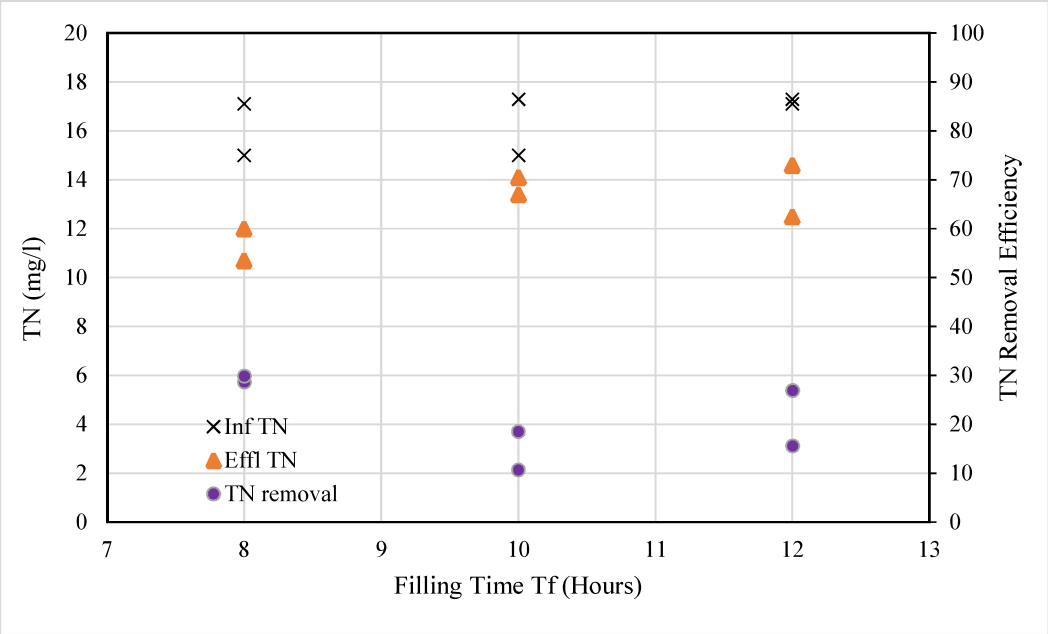

Figure 5-10 TN removal with filling time in pre-denitrification process

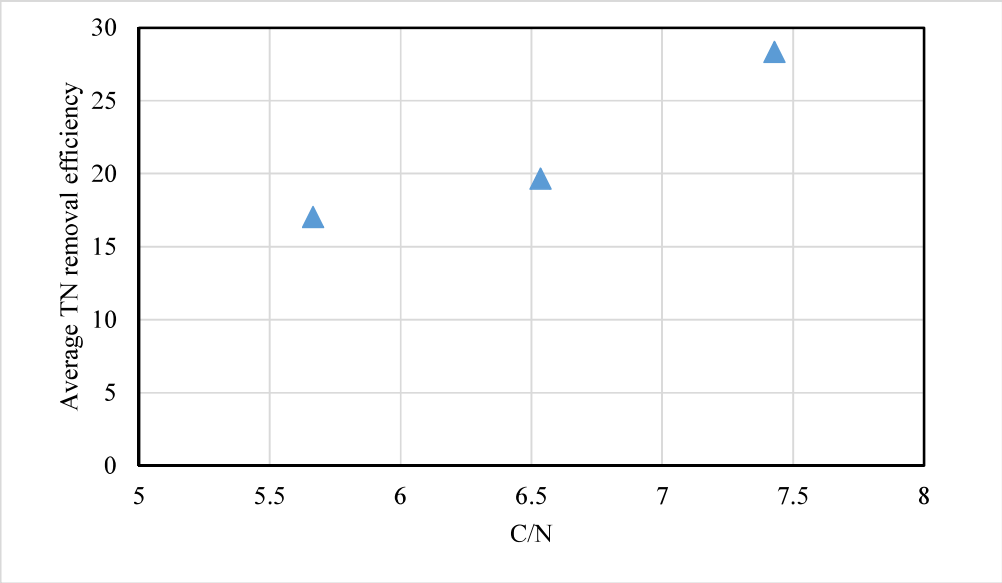

Figure 5-11 C/N ratio effect on TN removal in Pre-denitrification process

5.2.4 Total Kjeldahl Nitrogen (TKN)

5.2.4.1 TKN Removal in Post-anoxic Denitrification process

As we mentioned before, Post-anoxic denitrification process was presented using the settling time in which denitrification took place. Fig 5-12 illustrates the change in TKN removal with settling time (Ts) in post-denitrification process. The average removal efficiency reached 93.9%.

High removal efficiency was achieved in low C/TKN ratio of (4.6). Similar findings were observed by Blackburne et al. (2007). He achieved high removal efficiency (80%) achieved in low COD/TKN (8.6).

TKN in the influent is consisted of organically bonded nitrogen and ammonia nitrogen. The organic nitrogen (such as protein and urea) is readily converted to ammonia by hydrolysis process as we mentioned before. Equations 5-4 and 5-5 show the hydrolysis process and the enzymes that produced by the microorganisms to complete the process. Ammonia nitrogen is then converted to nitrite and then to nitrate by the process of nitrification. And as the environmental factors are optimum for nitrification to be occur such as pH levels; TKN will be eliminated from the wastewater without any difficulty.

$$\underset{\text{(Protein)}}{C-NH_2} + H_2O \xrightarrow{\text{Protease}} \underset{\text{(Organic C)}}{C-OH} + \underset{\text{(Ammonia)}}{NH_3} \qquad (5.4)$$

$$\underset{\text{(Urea)}}{H_2NCONH_2} + H_2O \xrightarrow{\text{Urease}} \underset{\text{(Bicarbonate)}}{HCO_3^-} + \underset{\text{(Ammonia)}}{NH_3} \qquad (5.5)$$

5.2.4.2 TKN Removal in Pre- anoxic Denitrification Process

Pre-denitrification process was presented using the filling time in which denitrification took place. Fig 5-13 shows the change in TKN removal with filling time in pre-denitrification process. An average removal efficiency of 94% was reached. Effluents were minimal.

And obviously there are no differences in the average TKN removals because as we mentioned that TKN has a considerable amount of organic nitrogen which is readily converted to ammonia nitrogen.

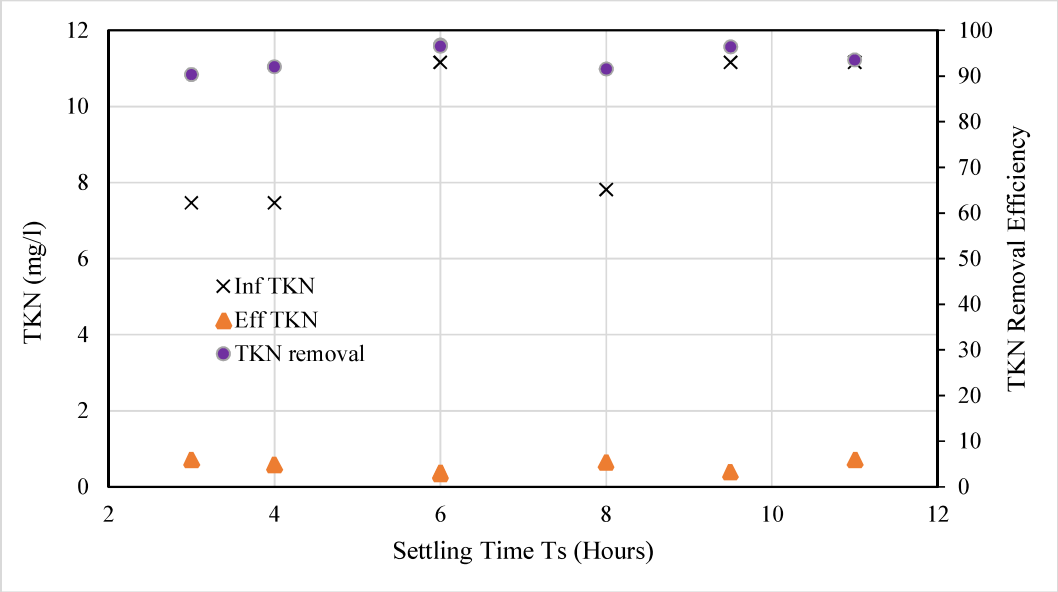

Figure 5-12 TKN removal with Settling time in Post-denitrification process

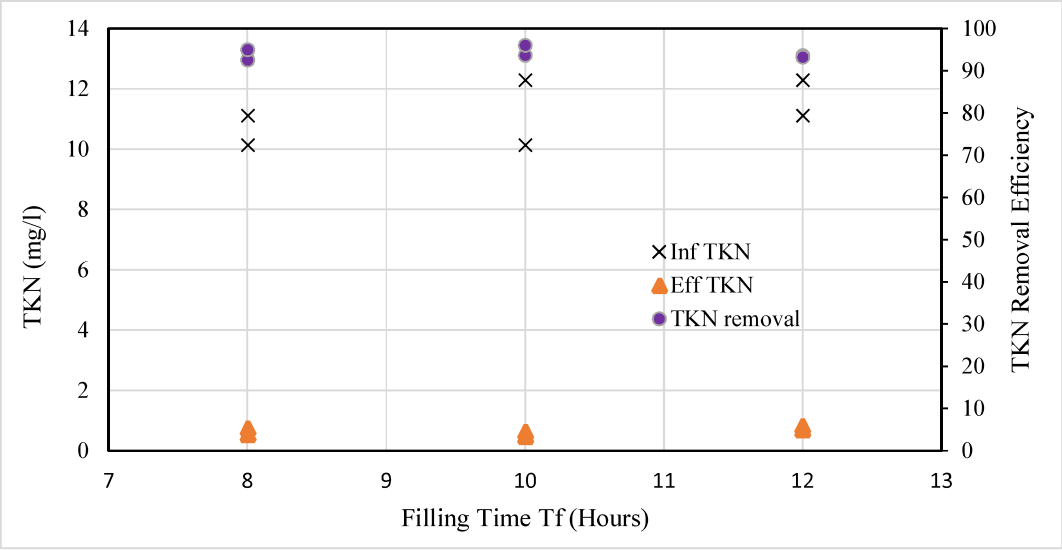

Figure 5-13 TKN removal with Filling time in Pre-denitrification process

5.2.5 Ammonia Nitrogen ($NH_4^+ - N$)

5.2.5.1 Ammonia Nitrogen Removal in Post- anoxic Denitrification Process

Ammonia-Nitrogen is the part of nitrogenous compounds which consumed in the nitrification zone, we did not notice obvious trend between ammonia removal and settling time (anoxic zone) in Fig 5-14 because ammonia-nitrogen is consumed in the nitrification zone. The average removal efficiency of the post-denitrification process is 91.2% which is significantly high. However, this value is relatively low in comparison with 98% removal for NSTP.

Achieving such removal efficiency was by maintaining the DO to be more than 3.0 mg/l in the nitrification zone, and temperature around 27^{o}C.

5.2.5.2 Ammonia Nitrogen Removal in Pre- anoxic Denitrification Process

Fig 5-15 shows the removal efficiencies of ammonia-nitrogen for pre-denitrification process. Again, no obvious trend of increasing or decreasing with respect to the filling time was observed, because ammonia-nitrogen is utilized in the nitrification zone as we mentioned before. The average removal efficiency of Ammonia was 93.2%.

Slightly higher average removal efficiency in the pre-denitrification process (93.2%) than post-denitrification (91.2%) was observed.

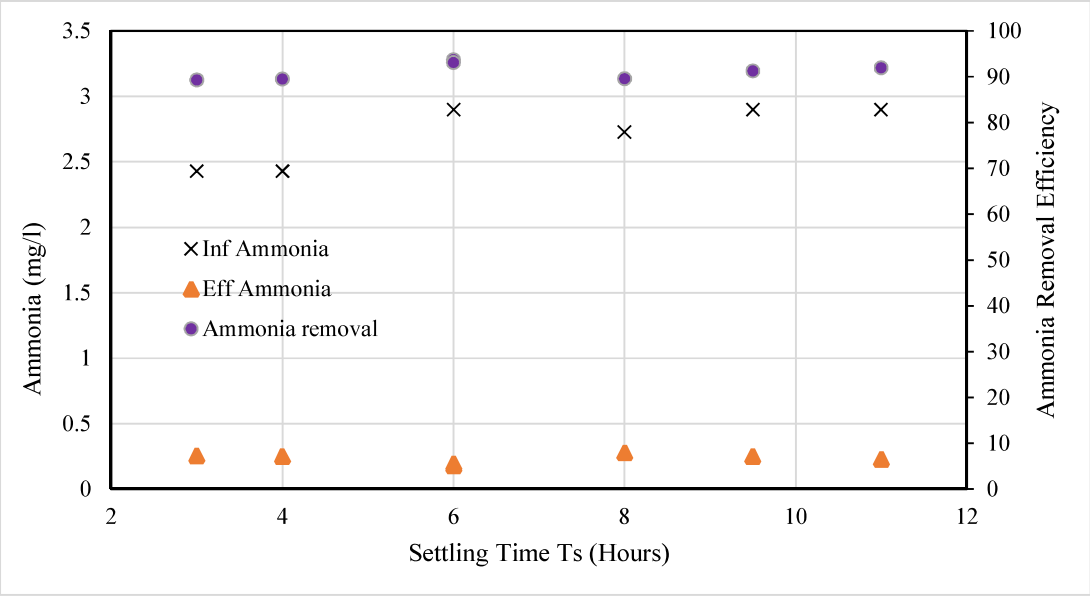

Figure 5-14 Ammonia removal with Settling time in Post-denitrification process

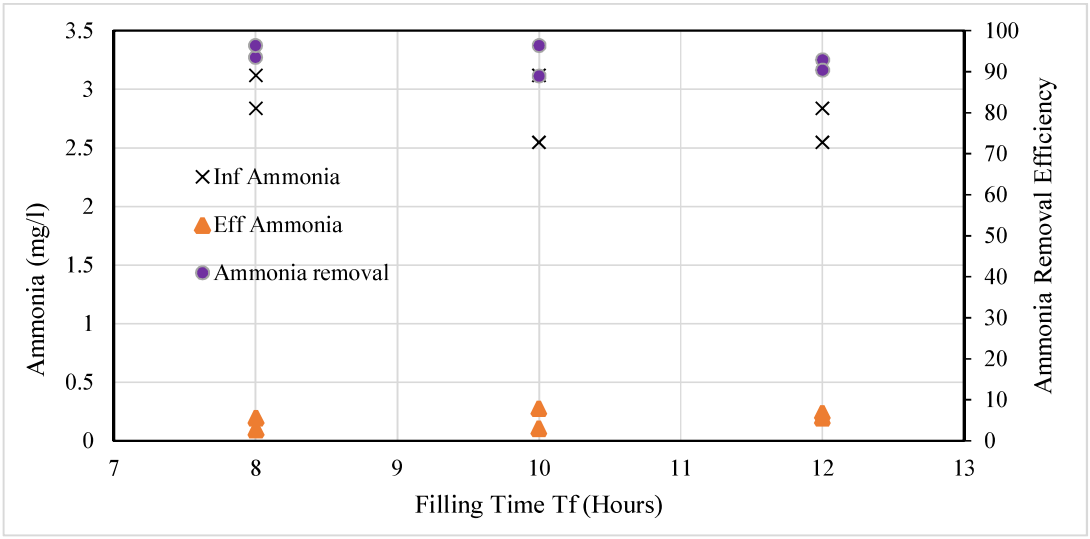

Figure 5-15 Ammonia removal with Filling time in Pre-denitrification process

5.2.6 pH

5.2.6.1 pH in Post- anoxic Denitrification Process

Fig 5-16 illustrates the pH change during post-denitrification process. The influent pH values were between 7.16 and 7.31 which were slightly in the alkaline side. An optimum values for overall nitrification reactions should be in the alkaline side (Bhavender et al. 1976) [10]. The nitrification reactions generally lower the pH of the system, and the system needs to be buffered. However, that is not considered when the system is designed for denitrification as well, because denitrifying organisms are less affected by pH change than nitrifying organisms. We recognized an increase in the effluent pH values which in Fig 5-18. And de-nitrification process generally raise the value of pH (Metcalf and Eddy 2003) [1].

5.2.6.2 pH in Pre- anoxic Denitrification Process

Fig 5-17 shows the pH change during pre-denitrification process. The influent pH values were between 7.39 and 7.68 which were in the alkaline side as well, and the optimum values for overall nitrification reactions must be in the alkaline side (Bhavender et al. 1976) [10].

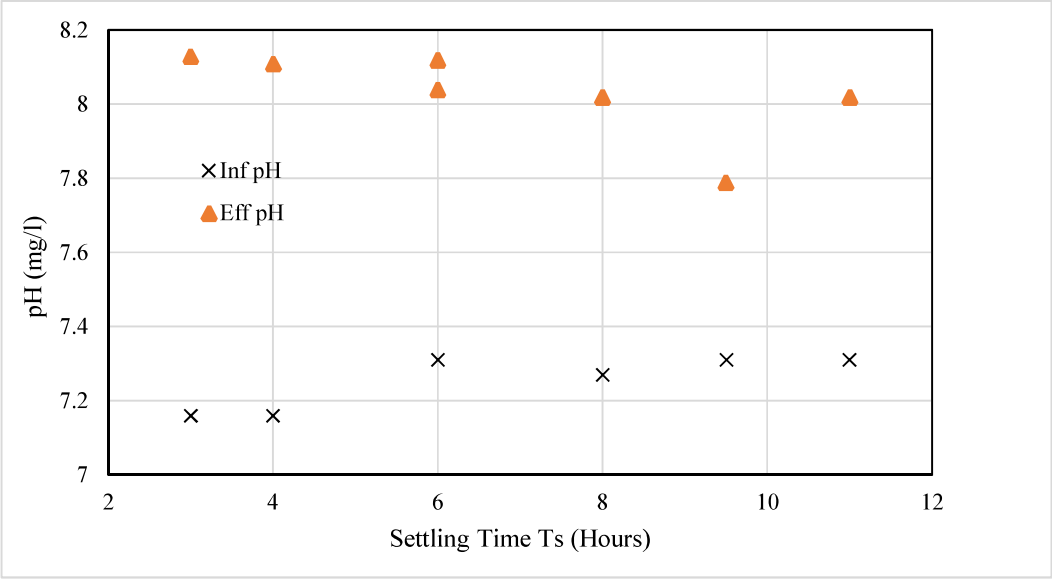

Figure 5-16 pH with settling time in Post-denitrification process

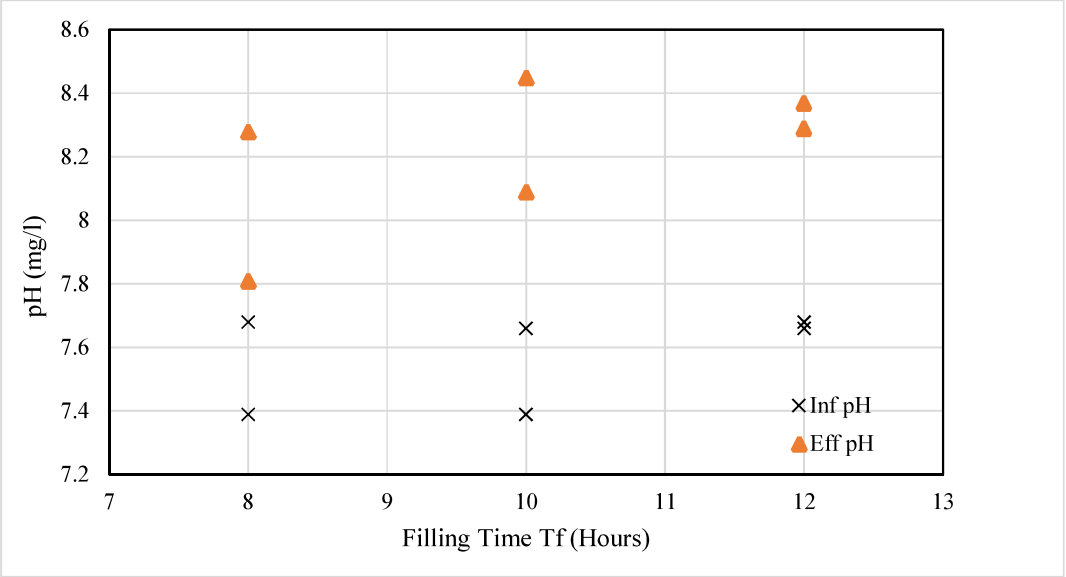

Figure 5-17 pH with filling time in Pre-denitrification process

5.2.7 Chemical Oxygen Demand (COD)

5.2.7.1 COD Removal in Post- anoxic Denitrification Process

Fig 5-18 illustrates the change in COD removal efficiency with respect to settling time (Ts) in the post-denitrification process. It is clear that the COD removal efficiency increases with the increase of settling time. The reason for this, is that the chemical oxygen demand is consumed in the denitrification process as a carbon source and electron donor, and since the settling time represent the time of the anoxic zone in which denitrification takes place, that means as the latter increases the COD removal efficiency increases. An average removal efficiency of 73.8% was achieved in post-denitrification process as shown in Fig 5-18.

5.2.7.2 COD Removal in Pre-anoxic Denitrification Process

Fig 5-19 shows the effect of C/N ratio on the removal of COD during the pre-denitrification process. Although the average C/N = 6.5, which is relatively low compared to the typical C/N of 10, it was clearly shown that as C/N ratio increases; COD removal increases. These findings can be justified from (Daekun et al. 2008) as they stated that COD removal can be achieved in low C/N ratio (1.1 to 7.4) [25].

This is because the dissimilating bacteria uses the carbon as electron donor source, so the potential for this bacteria will increase when available carbon is there to denitrify nitrate, allowing a reinforcing loop to occur again (i.e. more COD will be consumed) and COD removal will increase as a result. An average removal efficiency of 65.1 % was achieved in pre-denitrification process as illustrated in Fig 5-20.

Again one can say that post-denitrification process is better than pre-denitrification process in terms of COD removal.

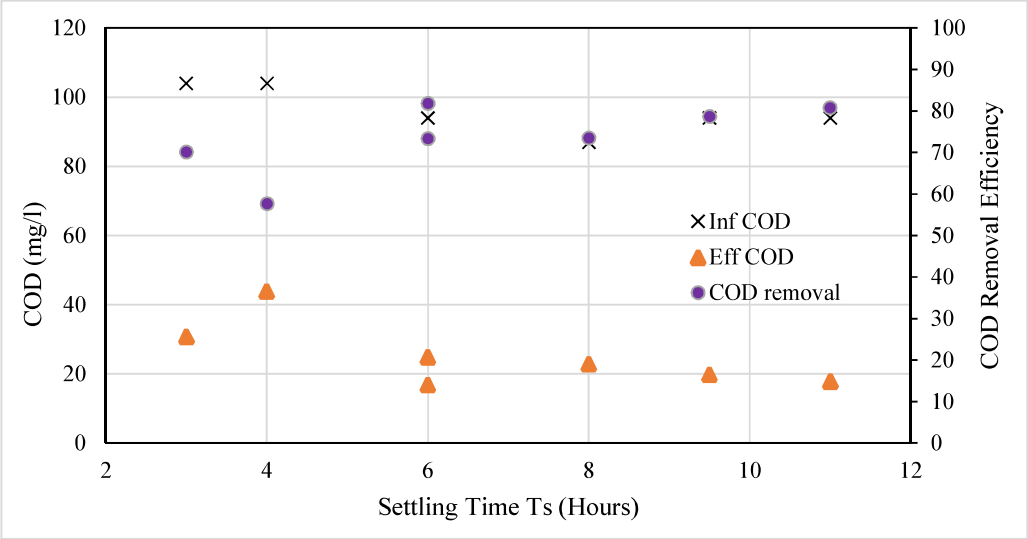

Figure 5-18 COD removal with Settling time in Post-denitrification process

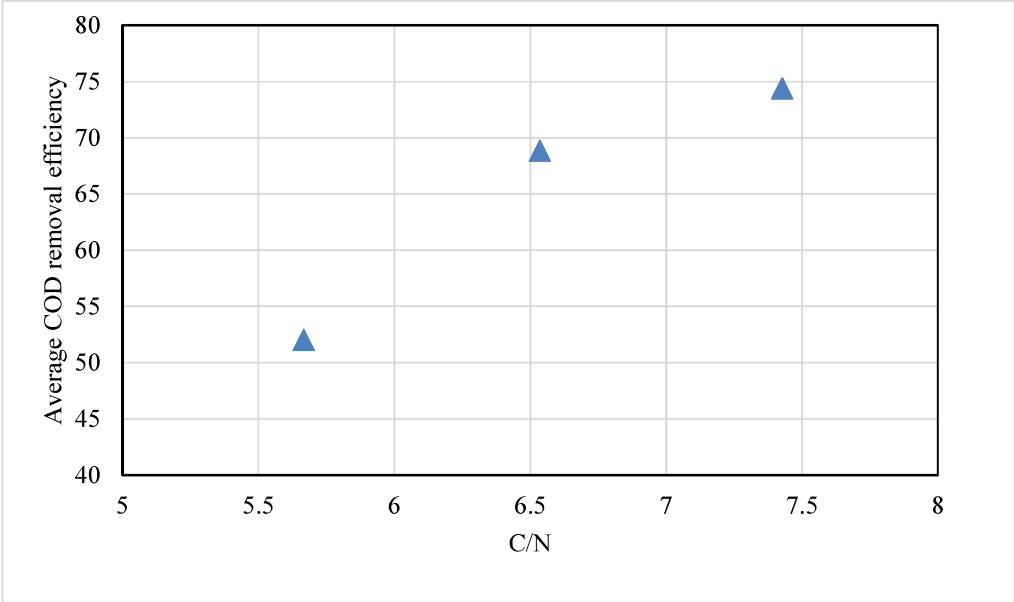

Figure 5-19 COD removal with C/N ration in Pre-denitrification process

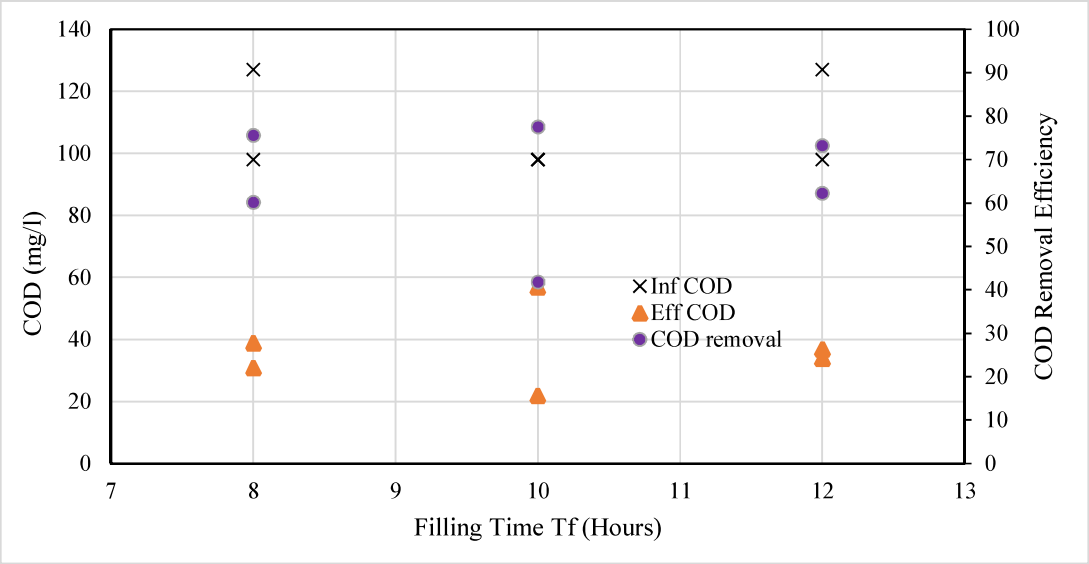

Figure 5-20 COD removal with C/N ratio in Pre-denitrification process

5.2.8 Turbidity

Turbidity removal is illustrated in Fig 5-21, the SBR process reached maximum Turbidity removal efficiency of 98% and average removal efficiency of 96%, which means that the colloidal and the residual suspended matter were minimal in the treated effluent. The consistency of the readings of the influent and effluent Turbidity indicates that no presence of air bubbles in the influent and treated effluent, since the air bubbles make the records inconsistent and erroneous turbidity readings occur. Because turbidity is affected by the light-absorbing properties of the suspended materials which widely vary from an experiment to another, and because it is also affected by the source of light and the method of measurement, it is almost impossible to compare our results to any results in the literature. However, they are still comparable to NSTP readings because the source of water is the same. The average turbidity removal efficiency of NSTP is 96.8%. In comparison to that, the turbidity removal efficiency of the SBR process is relatively high (96.0%), indicating that the SBR process has a significant turbidity removal efficiency.

The first seven batches which treated using post-denitrification relatively achieved lower values of effluent turbidity in comparison with the latter batches. The attribution to this is the higher settling times given in the post-denitrification process compared to the pre-denitrification process, giving the colloidal solids and the suspended solids enough time to accumulate and settle.

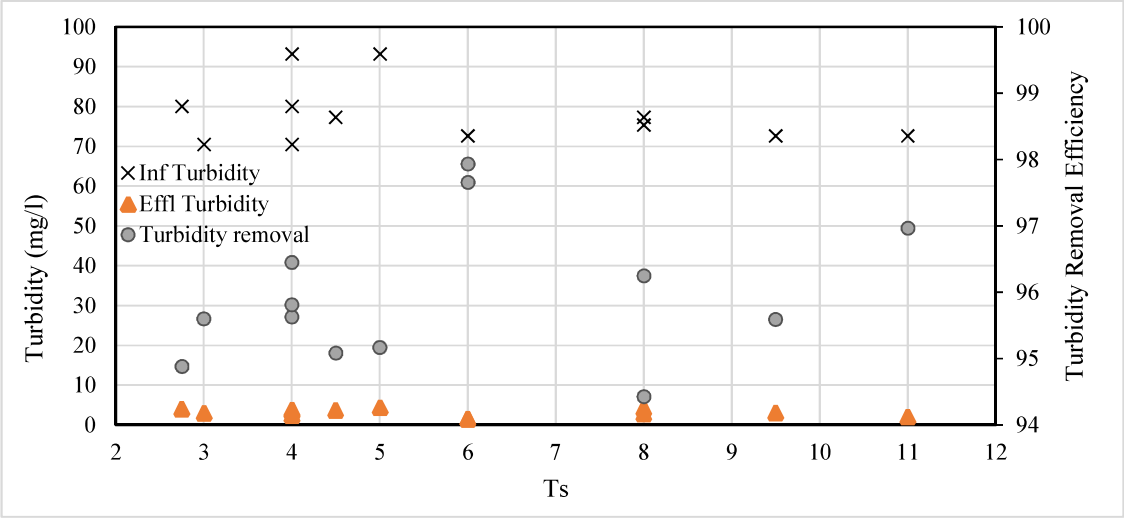

Figure 5-21 Turbidity removal with Settling time

5.2.9 Total Suspended Solids (TSS)

Fig 5-22 shows the influent, effluent and removal efficiency of TSS with respect to settling time (Ts). It is clear that the effluent concentrations were relatively low ranging between 3.6 mg/l and 10.6 mg/l. Unlike other water characteristics, TSS lacks a fundamental basis, yet it is considered one of the two universally used effluent standards along with BOD. TSS is generally affected by number of factors such as the type and pore size of the filter paper. Since all of the effluent TSS concentrations are relatively low values, it indicates that the measures were accurate and no auto-filtration took place. In comparison to the existing wastewater treatment plant (NSTP), the average TSS removal efficiency of the SBR in the post-denitrification process was 98.0% in comparison to 96.8% (for NSTP) which indicates the significance of SBR process in TSS removal.

No significant difference between post and pre denitrification processes are observed. The reason is that TSS depends on settling characteristics and settling time since it is the last phase before decanting.

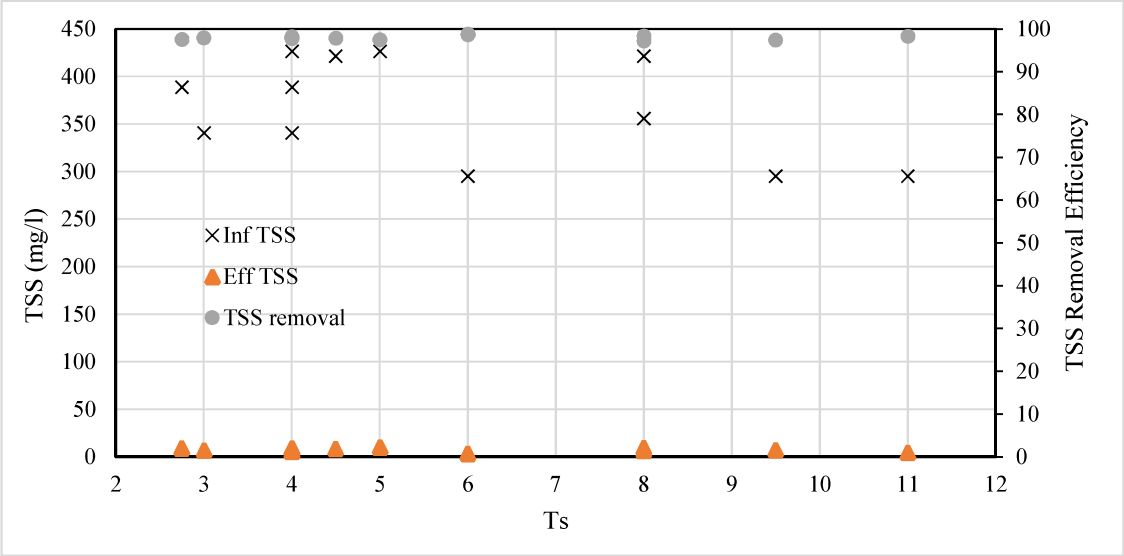

Figure 5-22 TSS removal with Settling time

5.3 Comparison between NSTP Activated Sludge Process and SBR Process

Since post-anoxic denitrification process proved to be the best choice for nitrogen removal and COD removal, let us make a comparison between it and the average removal efficiency of the conventional activated sludge process (i.e. NSTP). As shown in Table (5-2), the removal efficiency for NSTP is 65.8% and for SBR system is 50% and for ammonia and COD they were (91.2%, 75.3%) and (97.4%, 73.8%) for NSTP and SBR respectively.

The SBR process proved to be a very efficient technique in the removal of Turbidity as we have seen before. The average removal efficiency of Turbidity is 96.4% for post-anoxic comparing to average removal efficiency of NSTP process (96.2%). Also for TSS removal efficiency the SBR process achieves 98.0% removal efficiency with comparison to 96.7% as average removal efficiency of NSTP process.

Therefore, the pilot plant SBR process can be considered very suitable and considering the economic aspects and compactness of the SBR system, one can say it can be used as alternative to the centralized wastewater treatment plants.

Table 5-2 Comparison between NSTP and SBR processes

Average Removal Efficiency	North Sewage Treatment Plant NSTP	Sequencing Batch Reactor SBR
TN	65.8	50
Ammonia	91.2	97.4
COD	75.3	73.8
Turbidity	96.8	96.0
TSS	97	98

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

1- The performance assessment of the North Sewage Treatment Plant (NSTP) was found to be consistent with a well maintained secondary activated sludge process.

2- The performance assessment of NSTP gave a unique opportunity to study the treatment of low strength domestic wastewater not commonly encountered in actual situations.

3- Denitrification/nitrification sequencing batch reactor (SBR) is successfully established to investigate the performance of the treatment of municipal wastewater in Doha district in Dhahran area of the Eastern Province (NSTP). The SBR techniques is mainly used for the treatment of industrial wastewater as small-scale package plants.

4- Untreated samples were obtained to represent the influent and effluent of the NSTP for the whole study period of four months. The characterization of the municipal wastewater (WW) consisted of pH, Turbidity, Total Suspended Solids (TSS) chemical oxygen demand (COD), biochemical oxygen demand in 5 days (BOD5), Nitrate, and Ammonia were successfully achieved.

5- Two scenarios were performed, pre-anoxic and post-anoxic to study removal of nitrogen. During the 4 months of operation of the study, satisfactory and stable removal efficiencies of targeted parameters were achieved. The results showed

that post-anoxic denitrification was more efficient in the removal of Total nitrogen (50%) than pre-anoxic denitrification (21.7%). However, for the removal of Total Kjeldhal nitrogen the removal efficiency was almost the same around 94%. For Ammonia removal, the two scenarios gave similar results around 92% removal. As far as other parameters were concerned, similar results were obtained from the two scenarios. COD removal efficiency reached around 73%, turbidity averaged around 96%, and Total Suspended Solids averaged around 98%.

6- SBR processes can be used to treat municipal wastewater since the removal efficiencies of all parameters tested were comparable with the existing NSTP.

7- Post-anoxic denitrification scenario was found to be better than pre-anoxic denitrification scenario for the removal of total nitrogen (TN).

8- Post-anoxic scenario achieves better removal of chemical oxygen demand (COD) Pre-anoxic scenario for the SBR process.

9- Removal efficiencies for parameters such as, turbidity, total suspended solids (TSS), ammonia, and total kjeldhal nitrogen (TKN) were similar for post-anoxic and pre-anoxic scenarios.

6.2 RECOMMENDATIONS

In future studies and in order to increase the efficiency of this technique, it can be combined with other techniques such as membranes to establish membranes bioreactor MBR.

The use of external carbon source can be investigated in denitrification process to enhance to nitrogen removal.

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Appendix A, Experimental Results of NSTP Assessment

Table (A-1): 16 Weeks Influent Experimental Results of North Sewage Treatment Plant NSTP

Weeks	Sample ID	pH	Conductivity	Turbidity	TSS	NH4	D COD	T COD	BOD	DO	Nitrite	Phosphorus
			ms/cm	NTU	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l
1	B1	7.7	5.1	68.6	369.6	14.6	45.4	211.0	64.0	3.0	0.0	2.2
2	C1	8.1	4.0	46.4	319.8	19.8	58.4	205.0	76.0	4.0	0.1	4.2
3	D1	7.6	4.4	75.8	231.6	17.2	54.5	249.0	48.0	6.2	0.0	3.1
4	E1	7.6	4.5	56.1	178.8	21.8	58.8	195.0	68.0	7.2	0.1	1.4
5	F1	7.2	5.1	132.0	189	15.3	546.0	69.1	136.0	8.7	0.1	2.7
6	G1	7.3	4.9	114.0	221.67	19.7	87.6	242.0	82.0	0.8	0.0	2.3
7	H1	7.2	5.0	67.9	343.5	7.0	63.5	155.0	66.0	4.6	0.0	4.5
8	I1	7.1	4.2	44.5	103.2	12.6	55.1	68.0	78.0	4.3	0.1	0.3
9	J1	7.2	3.4	57.8	381.6	18.9	63.3	216.0	86.0	1.9	0.0	2.4
10	K1	7.3	4.6	34.6	243.3	15.7	67.1	119.0	52.0	1.3	0.0	3.1
11	L1	7.3	4.5	103.0	193.8	22.9	77.7	177.0	144.0	3.7	0.0	0.5
12	M1	7.5	4.8	187.0	294	20.3	128.0	359.0	88.0	5.5	0.1	0.6
13	N1	7.7	5.0	78.2	252.3	23.3	69.4	180.0	68.0	5.0	0.1	1.5
14	O1	7.4	2.3	103.0	299.4	11.2	74.2	175.0	68.0	5.0	0.1	1.2
15	P1	7.3	5.2	53.2	84.9	24.0	84.2	155.0	60.0	6.3	0.0	3.8
16	Q1	7.5	5.2	78.8	190.8	35.0	68.0	98.0	94.0	1.8	0.0	2.0

Table (A-2): 16 Weeks Secondary Effluent Experimental Results of North Sewage Treatment Plant NSTP

Weeks	Sample ID	pH	Conductivity	Turbidity	TSS	NH4	D COD	T COD	BOD	DO	Nitrite	Phosphorus
			ms/cm	NTU	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l
1	B2	7.9	5.2	2.3	2.3	0.0	22.9	23.6	10.0	1.0	0.0	1.6
2	C2	8.2	4.5	1.4	2.4	0.0	18.5	24.2	6.0	1.0	0.0	2.5
3	D2	7.4	4.3	1.8	2.35	0.2	20.7	22.5	14.0	4.4	0.0	1.6
4	E2	8.0	4.4	1.2	2.39	0.0	13.8	15.4	4.0	3.5	0.0	1.1
5	F2	7.6	5.1	1.6	2.31	0.0	19.4	22.7	18.0	6.5	0.0	0.9
6	G2	7.6	4.9	1.9	2.34	0.1	30.2	43.2	14.0	3.2	0.0	1.2
7	H2	7.5	4.7	1.9	2.4	0.1	51.6	64.5	18.0	3.9	0.0	4.4
8	I2	7.5	3.9	2.7	2.33	0.0	35.1	42.9	12.0	3.9	0.0	0.2
9	J2	7.5	4.3	1.7	2.38	0.0	20.0	38.5	4.0	3.4	0.0	0.7
10	K2	7.6	4.1	2.9	2.37	0.4	47.2	94.2	2.0	3.8	0.0	2.9
11	L2	7.5	4.2	4.5	2.39	0.3	28.8	71.9	8.0	5.2	0.0	0.6
12	M2	7.6	4.3	4.9	2.35	1.6	33.8	49.0	2.0	5.3	0.1	0.3
13	N2	7.7	5.5	5.1	2.4	0.3	25.2	36.9	4.0	5.5	0.0	0.1
14	O2	7.7	4.3	1.4	2.33	0.1	40.3	66.3	10.0	5.3	0.0	0.5
15	P2	7.1	35.5	4.4	2.38	0.2	30.0	35.7	4.0	5.3	0.0	0.0
16	Q2	7.5	5.0	2.4	2.37	0.4	36.0	56.0	8.0	4.8	0.0	1.0

- B1, C1, D1 etc. are influent samples
- B2, C2, D2, etc. are secondary effluent samples

Appendix B, Process Design for the Pilot Plant SBR

Overview

The design of SBR was adopted from Metcalf and Eddy [1], it consists of seventeen steps as shown in the design. The influent data was collected and considered as input data for the design, and the outputs of the design were used in the experimental setup.

Sequencing Batch Reactor Process Design Calculations

The process consisted of 17 detailed steps divided into two sections. The first section in which BOD removal and nitrification calculations done and the second section in which the denitrification calculations done.

The first section: BOD removal and nitrification

The following table, includes the main parameters for influent and some effluent requirements for the first sample S1.

Table (B-1): Wastewater characterizations of the First Sample (S1)

Constituent	Concentration mg/l
BOD	80.00
sBOD	70.00
COD	104.00
sCOD	75.00
rbCOD	17.16
TSS	340.80
VSS	236.00
TKN	7.47
NH4-N	2.43
TP	2.02
Alkalinity	310.00 as CaCO3
bCOD/BOD	1.60
Ne	0.50
DO	4.50

Design conditions and assumptions:

No. of Tanks = 1 tanks

Total liquid depth when full = 0.70 m

Decant depth = 30.0 %

SVI = 150.00 mg/l

NO_x = 80.0% of TKN

Each sample is treated in couple of batches, and the design sheet is for one batch, and the rest of calculations and results are in the appendixes.

Table (B-2): Design Times for the First Batch of First Sample

tA	4.00	h
tS	3.00	h
tD	0.30	h
tF	0.75	h

Table (B-3): Kinetic coefficients for heterotrophic bacteria at 20° C

Coefficient	Unit	Range	Typical value
μ_m	g VSS/g VSS.d	3.0-13.2	6.00
K_S	g Bcod/m^3	5.0-40.0	20.00
Y	g VSS/g bCOD	.30-.50	0.40
k_d	g VSS/g VSS.d	.06-.20	0.12
f_d	Unitless	.08-.15	0.15
Θ values			
μ_m	Unitless	1.03-1.08	1.07
k_d	Unitless	1.03-1.08	1.04
K_S	Unitless	1.00	1.00

Table (B-4): Kinetic coefficients for nitrification process at 20 °C

Coefficient	Unit	Range	Typical value
μ_m	g VSS/g VSS.d	0.20-.90	0.75
K_n	g Bcod/m^3	0.5-1.0	0.74
Y	g VSS/g bCOD	0.10-0.15	0.12
k_{dn}	g VSS/g VSS.d	0.05-0.15	0.08
K_o	Unitless	0.40-0.60	0.50
Θ values			
μ_m	Unitless	1.06-1.123	1.07
K_n	Unitless	1.03-1.123	1.05
k_{dn}	Unitless	1.03-1.08	1.04

Q =1.63 m^3/d

Temperature = 26.00 ℃

1. Wastewater characteristics needed for process design.

a) bCOD

$$bCOD = 1.6(BOD) \qquad (B.1)$$

= 128.00 g/m^3

b) Wastewater characteristics needed for process design

bpCOD/pCOD = 0.55

nbVSS = 105.793 g/m^3

c) iTSS

$$iTSS = TSS_o - VSS_o \qquad (B.2)$$

$= 104.800\ g/m^3$

2. SBR operating cycle

Filling time = 0.75 h

Total cycle time Tc = 8.05 h

Number of cycles/ tank.d = 2.98

Total number of cycles/d = 2.98 cycles/d

Fill volume/cycle = 0.08 m^3/fill

3. Fill fraction per cycle (Vf/Vt) allowed and compare to selected value of 0.30

a) Develop a mass balance based on solids in the reactor

Mass of solids at full volume = Mass of settled solids

$$V_T X = V_s X_s \qquad (B.3)$$

VT = total volume m^3

X = MLSS concentration at full volume g/m^3

Vs = settled volume after decant m^3

Xs = MLSS concentration in settled volume g/m^3

b) Solve mass balance and determine the fill fraction/ cycle.

i. Estimate Xs based on an assumed SVI value of 150.00 mg/l

Xs= 6666.67 g/m^3

X= 3500.00 g/m^3

ii. Determine the settled fraction.

$$\frac{Vs}{V_T} = \frac{X}{X_s} = 0.53$$

Provide 20 percent liquid above the sludge blanket so that solids not removed by the decanting mechanism 0.63

iii. Determine the fill fraction

$$V_F + V_S = V_T \qquad (B.5)$$

Where V_F = fill volume, m^3

$$\frac{V_F}{V_T} + \frac{V_S}{V_T} = 1.0 \qquad (B.6)$$

VF/VT = 0.37

Selected VF/VT = 0.30

4. Overall hydraulic retention time HRT

Full liquid depth = 0.70 m

Decant depth = 0.21 m

VT = 0.27 m^3/tank

Over all HRT = 2.50 h

5. SRT

a) PX,TSS

$$(P_{X,TSS})SRT = \frac{Q\,Y\,(S_o - S)SRT}{(1+(k_d SRT))0.85} + Q\,(nbVSS)SRT + \frac{QY_n(NO_X)SRT}{(1+(k_{dn} SRT))0.85} +$$

$$\frac{f_d k_d QY((S_o - S))SRT^2}{(1+(k_d SRT))0.85} + Q(TSS_o - VSS_o) \qquad (B.7)$$

$$(P_{X,TSS})\ SRT = V\ (X_{MLSS}) \qquad (B.8)$$

= 933.33 g

b) Develop input data to solve the above relationship for SRT

nbVSS = 105.80 g/m^3

Assume $S_o \approx S_0 - S\ S_o = bCOD$ = 128.00 g/m^3

Q = 2.56 m^3

$iTSS_o = TSS_O - VSS_o$ = 104.80 g/m^3

NOx = 5.98 g/m^3

SRT = 1.388 d

6. MLVSS concentration

a) Determine XMLVSS

$$(P_{X,VSS})\ SRT = V_T\ (X_{MVLSS}) \qquad (B.9)$$

$$(P_{X,VSS}) * SRT = \frac{Q\,Y\,(S_o - S)SRT}{(1+(k_d SRT))} + Q\,(nbVSS)\ SRT + \frac{QY_n(NO_X)SRT}{(1+(k_{dn} SRT))} + \frac{f_d k_d QY((S_o - S))SRT^2}{(1+(k_d SRT))} \qquad (B.10)$$

PX,VSS SRT = 533.18 m^3.g/m^3

XMLVSS = 1999.43 g/m^3

b) Determine the fraction of MLVSS

XMLVSS/XMLSS = 0.57

7. Amount of NH4-N Oxidized (NOx)

$$NO_X = TKN_o - N_e - 0.12P_{X,bio}/Q \qquad (B.11)$$

$$P_{X,bio} = \frac{Q\,Y\,(S_o - S)}{(1+(k_d SRT))} + \frac{QY_n(NO_X)}{(1+(k_{dn} SRT))} + \frac{f_d k_d QY((S_o - S))SRT}{(1+(k_d SRT))} \qquad (B.12)$$

PX,bio = 113.29 g/d

= 0.11 kg/d

NOx = 1.66 g/m^3

8. Check for the degree of nitrification to determine Whether NH4-N will be removed a level of 0.10 g/m^3 in a 4.00 h aeration period

a) Determine the amount of oxidizable N available

NOx = 2.06 g/m^3

Oxidizable NH4-N added/cycle:

VF(NOx) = 0.13 g/fill

NH4-N remaining before fill = Vs(Ne)

Ne = 0.50 g/m^3

Vs(Ne) = Ne (V-VF) = 0.09 g

Total oxidizable N at beginning of cycle = 0.23 g

Initial concentration = No = 0.85 g/m^3

b) Determine the reaction time.

The react time (aeration) after fill to achieve the desired NH4-N concentration can be calculated. First, the nitrifier concentration must be determined.

$$K_n \ln \frac{N_o}{N_t} + (N_o - N_t) = X_n \left(\frac{\mu_{mn}}{Y_n}\right)\left(\frac{DO}{K_o + DO}\right) t \qquad (B.13)$$

i. Nitrifier concentration

$$X_n = \frac{Q(Y_n)(NO_x)SRT}{(1+(k_d)SRT)V} \quad (B.14)$$

X_n = 0.69 g/m^3

$\mu_{m,T\,°C} = \mu_m\, \theta^{T-20}$ = 1.13

$K_{n,T\,°C} = k_{20}\, \theta^{T-20}$ = 1.01

K_o = 0.50

ii. Determine the time for reaction

Solve for t for No = 0.85 g/m^3

Ne = 0.50 g/m^3

t = 0.15 days

= 3.63 h

c) Determine the aeration time

Required aeration time = 3.63h

Aeration time selected was = 4.00 h

Therefore Aeration time is OK

9. Decant pumping rate:

Decant volume = Fill volume

Pumping rate = 0.004 m^3/min

10. Oxygen required/tank:

$$R_o = Q(S_o - S) - 1.42P_{X,bio} + 4.33Q(NO_x) \qquad (B.15)$$

R_o = 0.19 kg/d

Aeration time/cycle = 4.00 h

Number of cycles = 2.98

Total aeration time = 13.04 h/d

Average oxygen transfer rate = 0.02 kg/h

Safety factor = 2.00

Peak oxygen transfer = 0.03 kg/h

11. Sludge production

$$P_{x,TSS} = \frac{(V)(MLSS)}{SRT} \qquad (B.16)$$

= 0.67 kg/d

bCOD removed = 0.33 kg/d

BOD removed = 0.21 kg/d

Observed yield, gTSS/gBOD = 3.28 gTSS/gBOD

Observed yield, gVSS/gBOD = 2.66 gVSS/gBOD

Observed yield, gTSS/gbCOD = 2.05 gTSS/gbCOD

12. F/M and BOD volumetric loading

a. F/M

$$F/M = \frac{\text{kg BOD}}{\text{kg MLVSS.d}} = \frac{QS_o}{XV} \quad (B.17)$$

F/M = 0.38 g/g.d

a. BOD volumetric loading

$$L_{org} = \frac{\text{kg BOD}}{m^3.d} = \frac{QS_o}{V} \quad (B.18)$$

Lorg = 0.77 $kg/m^3.d$

13. Design summary for average conditions:

Table (B-5): Design summary sheet

Design parameter	Unit	Value
Average flow	m^3/d	2.56
Average BOD load	kg/d	0.21
Average TKN load	kg/d	0.02
Number of tanks	Number	1.00
Fill time	h	0.75
React time	h	4.00
Total aeration time	h	4.00
Settle time	h	3.00
Decant time	h	0.30
Cycle time	h	8.05
Total SRT	d	1.39
Tank volume	m^3	0.27
Fill volume/cycle	m^3	0.08
Fill volume/tank volume	Ratio	0.30
Decant depth	m	0.21
Tank depth	m	0.70
MLSS	g	3500.00
MLVSS	g	1999.43

F/M	g/g.d	0.38
Volumetric BOD load	kg/m^3.d	0.77
Decant pumping rate	m^3	0.004
Sludge production	kg/d	0.67
Observed yield	kgVSS/kgBOD	2.66
	kgTSS/kgbCOD	2.05
Average oxygen required/tank	kg/d	0.186
Total aeration time/d-tank	h	11.93
Average O2 transfer rate	kg/h	0.016

The second section: Nitrate removal (denitrification)

14. Nitrate produced (NOx):

a) Determine heterotrophic biomass produced

$$P_X = \frac{Q\,Y\,(S_o - S)}{(1+(k_d SRT))} + \frac{f_d k_d Q Y((S_o - S))SRT}{(1+(k_d))} \quad (B.19)$$

= 0.112 kg/d

b) Determine N synthesis

NOx = TKN-Nsyn- (NH_4-N)e

Nsyn = 0.013 kg/d

= 5.24 g/m^3

Effluent NH_4-N = 0.50 g/m^3

NOx produced = 1.73 g/m^3

15. Amount of nitrate left in mixed liquor after decant

Assume NO_3-N = 0 before aeration period.

a) Prepare nitrogen balance

NOx = 1.73 g/m^3

b) Determine NOx produced per cycle

g NO_3 produced per cycle = 0.139 g NOx/fill

NO_3-N concentration at end of aeration with tank full = 0.52 g/m^3

c) Determine nitrate remaining in SBR

Volume remaining after decant = 0.187 m^3

NO_3-N present = 0.52 g/m^3

= 0.097 g

16. SDNRb in fill period

SNDRb = 0.03 (F/M) + 0.029 = 0.03 g/g. d at 20℃ (B.20)

SNDRb At 26.00 ℃ = 0.047 g/g. d

17. NO3-N removal capacity during the fill period:

Active biomass = 0.108 kg/d

Active biomass concentration = 563.5 g/m^3

NOx = SDNRb (Xb)(VT) = 7.1 g/d

Fill time = 0.75 h

NOr at 0.75 h = 0.22 g

NO_3-N from step 15 = 0.097 g which is less than the capacity of NO_3-N removal.

Following the same approach for the rest of the wastewater samples and batches, the samples characterizations, times for the batches, and design results are found in appendix B.

Appendix C, SBR Process Design Results

Table (C-1): Design Times for the Second Batch of the First Sample (B12)

B12		
tA	3	h
tS	4	h
tD	0.3	h
tF	0.75	h

Table (C-2): Design summary sheet for the Second Batch of the First Sample (B12)

Design parameter	**Unit**	**Value**
Average flow	m^3/d	2.560
Average BOD load	kg/d	0.205
Average TKN load	kg/d	0.019
Number of tanks	Number	1.000
Fill time	h	0.750
React time	h	3.000
Total aeration time	h	3.375
Settle time	h	4.000
Decant time	h	0.300
Cycle time	h	8.050
Total SRT	d	1.387
Tank volume	m^3	0.267
Fill volume/cycle	m^3	0.080
Fill volume/tank volume	Ratio	0.300
Decant depth	m	0.210
Tank depth	m	0.700
MLSS	g/m^3	3500.000
MLVSS	g/m^3	1999.985
F/M	g/g.d	0.384
Volumetric BOD load	kg/m^3.d	0.768
Decant pumping rate	m^3/min	0.004
Sludge production	kg/d	0.673
Observed yield	kgVSS/kgBOD	2.661
	kgTSS/kgbCOD	2.053
Average oxygen required/tank	kg/d	0.189
Total aeration time/d-tank	h	10.062
Average O2 transfer rate	kg/h	0.019

Table (C-1): Design Times for the first Batch of the Second Sample

B21

tA	4	h
tS	8	h
tD	0.25	h
tF	1	h

Table (C-2): Design summary sheet for the first Batch of the Second Sample

Design parameter	**Unit**	**Value**
Average flow	m^3/d	1.920
Average BOD load	kg/d	0.127
Average TKN load	kg/d	0.015
Number of tanks	Number	1.000
Fill time	h	1.000
React time	h	4.000
Total aeration time	h	4.500
Settle time	h	8.000
Decant time	h	0.250
Cycle time	h	13.250
Total SRT	d	2.186
Tank volume	m^3	0.267
Fill volume/cycle	m^3	0.080
Fill volume/tank volume	Ratio	0.300
Decant depth	m	0.210
Tank depth	m	0.700
MLSS	g/m^3	3500.000
MLVSS	g/m^3	1316.901
F/M	g/g.d	0.361
Volumetric BOD load	kg/m^3.d	0.475
Decant pumping rate	m^3/min	0.005
Sludge production	kg/d	0.427
Observed yield	kgVSS/kgBOD	2.730
	kgTSS/kgbCOD	2.106
Average oxygen required/tank	kg/d	0.142
Total aeration time/d-tank	h	8.151
Average O2 transfer rate	kg/h	0.017

Table (C-1): Design Times for the First Batch of the Third Sample

B31

tA	4	h
tS	9.5	h
tD	0.4166	h
tF	0.85	h

Table (C-2): Design summary sheet for the First Batch of the Third Sample

Design parameter	**Unit**	**Value**
Average flow	m^3/d	2.260
Average BOD load	kg/d	0.339
Average TKN load	kg/d	0.025
Number of tanks	Number	1.000
Fill time	h	0.850
React time	h	4.000
Total aeration time	h	4.425
Settle time	h	9.500
Decant time	h	0.417
Cycle time	h	14.767
Total SRT	d	1.434
Tank volume	m^3	0.267
Fill volume/cycle	m^3	0.080
Fill volume/tank volume	Ratio	0.300
Decant depth	m	0.210
Tank depth	m	0.700
MLSS	g/m^3	3500.000
MLVSS	g/m^3	2683.723
F/M	g/g.d	0.474
Volumetric BOD load	kg/m^3.d	1.271
Decant pumping rate	m^3/min	0.003
Sludge production	kg/d	0.651
Observed yield	kgVSS/kgBOD	1.555
	kgTSS/kgbCOD	1.200
Average oxygen required/tank	kg/d	0.294
Total aeration time/d-tank	h	7.192
Average O2 transfer rate	kg/h	0.041

Table (C-1): Design Times for the Second Batch of the Third Sample

B32

tA	4	h
tS	11	h
tD	0.25	h
tF	0.5	h

Table (C-2): Design summary sheet for the Second Batch of the Third Sample

Design parameter	**Unit**	**Value**
Average flow	m^3/d	3.840
Average BOD load	kg/d	0.576
Average TKN load	kg/d	0.043
Number of tanks	Number	1.000
Fill time	h	0.500
React time	h	4.000
Total aeration time	h	4.250
Settle time	h	11.000
Decant time	h	0.250
Cycle time	h	15.750
Total SRT	d	0.824
Tank volume	m^3	0.267
Fill volume/cycle	m^3	0.080
Fill volume/tank volume	Ratio	0.300
Decant depth	m	0.210
Tank depth	m	0.700
MLSS	g/m^3	3500.000
MLVSS	g/m^3	2690.679
F/M	g/g.d	0.803
Volumetric BOD load	kg/m^3.d	2.160
Decant pumping rate	m^3/min	0.005
Sludge production	kg/d	1.133
Observed yield	kgVSS/kgBOD	1.593
	kgTSS/kgbCOD	1.229
Average oxygen required/tank	kg/d	0.455
Total aeration time/d-tank	h	6.476
Average O2 transfer rate	kg/h	0.070

Table (C-1): Design Times for the third Batch of the third Sample

B33

tA	4	h
tS	6	h
tD	0.283	h
tF	0.583	h

Table (C-2): Design summary sheet for the third Batch of the third Sample

Design parameter	**Unit**	**Value**
Average flow	m^3/d	3.300
Average BOD load	kg/d	0.495
Average TKN load	kg/d	0.037
Number of tanks	Number	1.000
Fill time	h	0.583
React time	h	4.000
Total aeration time	h	4.292
Settle time	h	6.000
Decant time	h	0.283
Cycle time	h	10.866
Total SRT	d	0.962
Tank volume	m^3	0.267
Fill volume/cycle	m^3	0.080
Fill volume/tank volume	Ratio	0.300
Decant depth	m	0.210
Tank depth	m	0.700
MLSS	g/m^3	3500.000
MLVSS	g/m^3	2689.577
F/M	g/g.d	0.690
Volumetric BOD load	kg/m^3.d	1.856
Decant pumping rate	m^3/min	0.005
Sludge production	kg/d	0.970
Observed yield	kgVSS/kgBOD	1.587
	kgTSS/kgbCOD	1.224
Average oxygen required/tank	kg/d	0.397
Total aeration time/d-tank	h	9.479
Average O2 transfer rate	kg/h	0.042

Table (C-1): Design Times for the Fourth Batch of the Third Sample

B34

tA	6	h
tS	6	h
tD	0.2	h
tF	0.583	h

Table (C-2): Design summary sheet for the Fourth Batch of the Third Sample

Design parameter	**Unit**	**Value**
Average flow	m^3/d	3.290
Average BOD load	kg/d	0.494
Average TKN load	kg/d	0.037
Number of tanks	Number	1.000
Fill time	h	0.583
React time	h	6.000
Total aeration time	h	6.292
Settle time	h	6.000
Decant time	h	0.200
Cycle time	h	12.783
Total SRT	d	0.964
Tank volume	m^3	0.267
Fill volume/cycle	m^3	0.080
Fill volume/tank volume	Ratio	0.300
Decant depth	m	0.210
Tank depth	m	0.700
MLSS	g/m^3	3500.000
MLVSS	g/m^3	2689.970
F/M	g/g.d	0.688
Volumetric BOD load	$kg/m^3.\mathrm{d}$	1.851
Decant pumping rate	m^3/min	0.007
Sludge production	kg/d	0.968
Observed yield	kgVSS/kgBOD	1.589
	kgTSS/kgbCOD	1.226
Average oxygen required/tank	kg/d	0.394
Total aeration time/d-tank	h	11.812
Average O2 transfer rate	kg/h	0.033

Table (C-1): Design Times for the First Batch of the Fourth Sample

B41

tA	4	h
tS	2.75	h
tD	0.25	h
tF	8	h

Table (C-2): Design summary sheet for the First Batch of the Fourth Sample

Design parameter	**Unit**	**Value**
Average flow	m^3/d	0.240
Average BOD load	kg/d	0.017
Average TKN load	kg/d	0.002
Number of tanks	Number	1.000
Fill time	h	8.000
React time	h	4.000
Total aeration time	h	4.000
Settle time	h	2.750
Decant time	h	0.250
Cycle time	h	15.000
Total SRT	d	19.232
Tank volume	m^3	0.267
Fill volume/cycle	m^3	0.080
Fill volume/tank volume	Ratio	0.300
Decant depth	m	0.210
Tank depth	m	0.700
MLSS	g/m^3	3500.000
MLVSS	g/m^3	510.328
F/M	g/g.d	0.123
Volumetric BOD load	kg/m^3.d	0.063
Decant pumping rate	m^3/min	0.005
Sludge production	kg/d	0.049
Observed yield	kgVSS/kgBOD	2.340
	kgTSS/kgbCOD	1.805
Average oxygen required/tank	kg/d	0.030
Total aeration time/d-tank	h	6.400
Average O2 transfer rate	kg/h	0.005

Table (C-1): Design Times for the Second Batch of the Fourth Sample

B42

tA	4	h
tS	4	h
tD	0.25	h
tF	10	h

Table (C-2): Design summary sheet for the Second Batch of the Fourth Sample

Design parameter	**Unit**	**Value**
Average flow	m^3/d	0.192
Average BOD load	kg/d	0.013
Average TKN load	kg/d	0.002
Number of tanks	Number	1.000
Fill time	h	10.000
React time	h	4.000
Total aeration time	h	4.000
Settle time	h	4.000
Decant time	h	0.250
Cycle time	h	18.250
Total SRT	d	24.294
Tank volume	m^3	0.267
Fill volume/cycle	m^3	0.080
Fill volume/tank volume	Ratio	0.300
Decant depth	m	0.210
Tank depth	m	0.700
MLSS	g/m^3	3500.000
MLVSS	g/m^3	484.273
F/M	g/g.d	0.104
Volumetric BOD load	kg/m^3.d	0.050
Decant pumping rate	m^3/min	0.005
Sludge production	kg/d	0.038
Observed yield	kgVSS/kgBOD	2.315
	kgTSS/kgbCOD	1.787
Average oxygen required/tank	kg/d	0.024
Total aeration time/d-tank	h	5.260
Average O2 transfer rate	kg/h	0.005

Table (C-1): Design Times for the First Batch of the Fifth Sample

B51

tA	6	h
tS	4.5	h
tD	0.33	h
tF	8	h

Table (C-2): Design summary sheet for the First Batch of the Fifth Sample

Design parameter	**Unit**	**Value**
Average flow	m^3/d	0.240
Average BOD load	kg/d	0.022
Average TKN load	kg/d	0.003
Number of tanks	Number	1.000
Fill time	h	8.000
React time	h	6.000
Total aeration time	h	6.000
Settle time	h	4.500
Decant time	h	0.333
Cycle time	h	18.833
Total SRT	d	12.263
Tank volume	m^3	0.267
Fill volume/cycle	m^3	0.080
Fill volume/tank volume	Ratio	0.300
Decant depth	m	0.210
Tank depth	m	0.700
MLSS	g/m^3	3500.000
MLVSS	g/m^3	1575.520
F/M	g/g.d	0.051
Volumetric BOD load	kg/m^3.d	0.081
Decant pumping rate	m^3/min	0.004
Sludge production	kg/d	0.076
Observed yield	kgVSS/kgBOD	2.854
	kgTSS/kgbCOD	2.202
Average oxygen required/tank	kg/d	0.034
Total aeration time/d-tank	h	7.646
Average O2 transfer rate	kg/h	0.004

Table (C-1): Design Times for the Second Batch of the Fifth Sample

B52

tA	6	h
tS	8	h
tD	0.25	h
tF	12	h

Table (C-2): Design summary sheet for the Second Batch of the Fifth Sample

Design parameter	**Unit**	**Value**
Average flow	m^3/d	0.160
Average BOD load	kg/d	0.014
Average TKN load	kg/d	0.002
Number of tanks	Number	1.000
Fill time	h	12.000
React time	h	6.000
Total aeration time	h	6.000
Settle time	h	8.000
Decant time	h	0.250
Cycle time	h	26.250
Total SRT	d	18.662
Tank volume	m^3	0.267
Fill volume/cycle	m^3	0.080
Fill volume/tank volume	Ratio	0.300
Decant depth	m	0.210
Tank depth	m	0.700
MLSS	g/m^3	3500.000
MLVSS	g/m^3	1555.183
F/M	g/g.d	0.035
Volumetric BOD load	kg/m^3.d	0.054
Decant pumping rate	m^3/min	0.005
Sludge production	kg/d	0.050
Observed yield	kgVSS/kgBOD	2.813
	kgTSS/kgbCOD	2.171
Average oxygen required/tank	kg/d	0.024
Total aeration time/d-tank	h	5.486
Average O2 transfer rate	kg/h	0.004

Table (C-1): Design Times for the First Batch of the Sixth Sample

B61

tA	4	h
tS	5	h
tD	0.3	h
tF	12	h

Table (C-2): Design summary sheet for the First Batch of the Sixth Sample

Design parameter	**Unit**	**Value**
Average flow	m^3/d	0.160
Average BOD load	kg/d	0.013
Average TKN load	kg/d	0.002
Number of tanks	Number	1.000
Fill time	h	12.000
React time	h	4.000
Total aeration time	h	10.000
Settle time	h	5.000
Decant time	h	0.300
Cycle time	h	21.300
Total SRT	d	17.840
Tank volume	m^3	0.267
Fill volume/cycle	m^3	0.080
Fill volume/tank volume	Ratio	0.300
Decant depth	m	0.210
Tank depth	m	0.700
MLSS	g/m^3	3500.000
MLVSS	g/m^3	1491.598
F/M	g/g.d	0.032
Volumetric BOD load	kg/m^3.d	0.048
Decant pumping rate	m^3/min	0.004
Sludge production	kg/d	0.052
Observed yield	kgVSS/kgBOD	3.311
	kgTSS/kgbCOD	2.554
Average oxygen required/tank	kg/d	0.023
Total aeration time/d-tank	h	11.268
Average O2 transfer rate	kg/h	0.002

Table (C-1): Design Times for the Second Batch of the Sixth Sample

B62

tA	4	h
tS	4	h
tD	0.3	h
tF	10	h

Table (C-2): Design summary sheet for the Second Batch of the Sixth Sample

Design parameter	**Unit**	**Value**
Average flow	m^3/d	0.192
Average BOD load	kg/d	0.015
Average TKN load	kg/d	0.002
Number of tanks	Number	1.000
Fill time	h	10.000
React time	h	4.000
Total aeration time	h	9.000
Settle time	h	4.000
Decant time	h	0.300
Cycle time	h	18.300
Total SRT	d	14.760
Tank volume	m^3	0.267
Fill volume/cycle	m^3	0.080
Fill volume/tank volume	Ratio	0.300
Decant depth	m	0.210
Tank depth	m	0.700
MLSS	g/m^3	3500.000
MLVSS	g/m^3	1502.237
F/M	g/g.d	0.038
Volumetric BOD load	kg/m^3.d	0.058
Decant pumping rate	m^3/min	0.004
Sludge production	kg/d	0.063
Observed yield	kgVSS/kgBOD	3.335
	kgTSS/kgbCOD	2.573
Average oxygen required/tank	kg/d	0.027
Total aeration time/d-tank	h	11.803
Average O2 transfer rate	kg/h	0.002

Appendix D, Experimental Setup Analysis Results

Table (D-1): Results of the 1st week experimental setup

Sample ID	DO	pH	Conductivity	Nitrate	Nitrite	TN	COD
	mg/l	mg/l	ms/cm	mg/l	mg/l	mg/l	mg/l
SA*	3.48	7.41	4.76	1.38	0.50	12.10	198.00
SA8**	4.30	8.20	4.65	1.81	0.52	17.40	73.00
SA16	4.90	8.17	5.08	1.58	0.43	17.20	106.00
SA24	5.05	8.18	4.45	1.38	0.55	16.30	68.00
SB	3.30	7.48	5.93	1.50	0.53	23.00	272.00
SB8	4.95	8.25	4.44	1.37	0.56	13.20	63.00
SB16	4.93	8.22	4.66	1.31	0.57	17.70	51.00
SB24	4.34	8.25	4.70	1.30	0.54	9.44	68.00
SC	1.95	7.35	4.39	1.30	0.51	19.70	94.00
SC8	3.90	7.69	4.54	1.27	0.50	3.59	65.00
SC16	4.23	8.10	4.70	1.27	0.54	17.30	58.00
SC24	4.08	7.94	4.44	0.58	0.11	17.50	62.00
SD	2.55	7.37	3.88	0.51	0.09	20.70	89.00
SD8	3.86	7.94	3.98	0.54	0.13	21.70	71.00
SD16	4.11	8.04	4.27	0.44	0.19	20.10	67.00
SD24	4.38	8.00	3.73	0.72	0.18	22.10	58.00
SE	2.56	7.30	3.60	0.61	0.13	18.20	80.00
SE8	5.58	7.87	3.75	0.61	0.21	13.10	71.00
SE16	4.85	7.92	4.56	0.59	0.20	15.80	52.00

SE24	4.51	7.71	4.12	0.60	0.20	16.00	26.40
SF	2.70	7.28	4.14	0.56	0.20	20.00	85.00
SF8	3.65	7.88	4.12	0.57	0.14	18.00	55.00
SF16	4.27	7.92	4.51	0.00	0.21	18.40	31.70
SF24	4.75	7.98	3.83	0.27	0.04	18.90	21.40

*The name of the collected wastewater sample.

**The number of the batch (8hours, 16hrs, 24hrs).

Appendix E, ARDUINO and Microcontroller Codes

Figure (E-1): Arduino interface

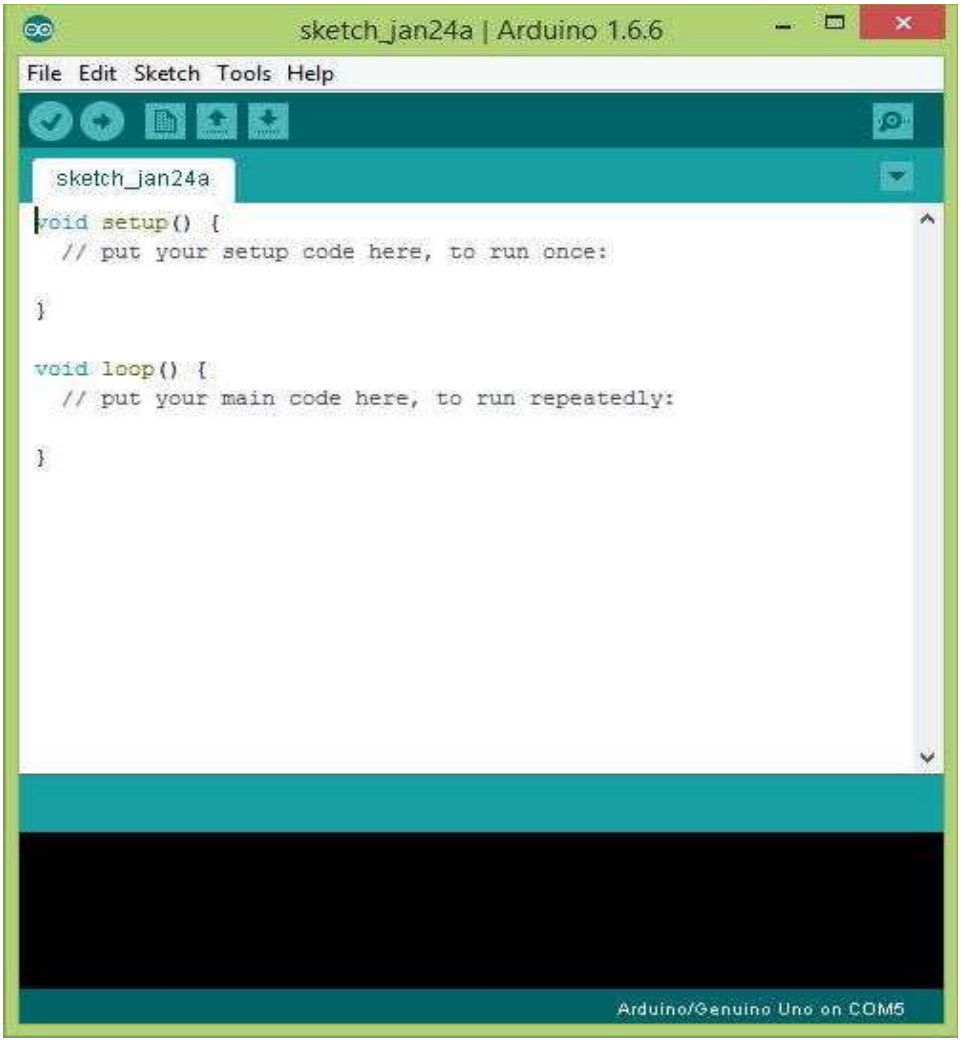


Figure (E-2): Arduino times data entry and pin Modes

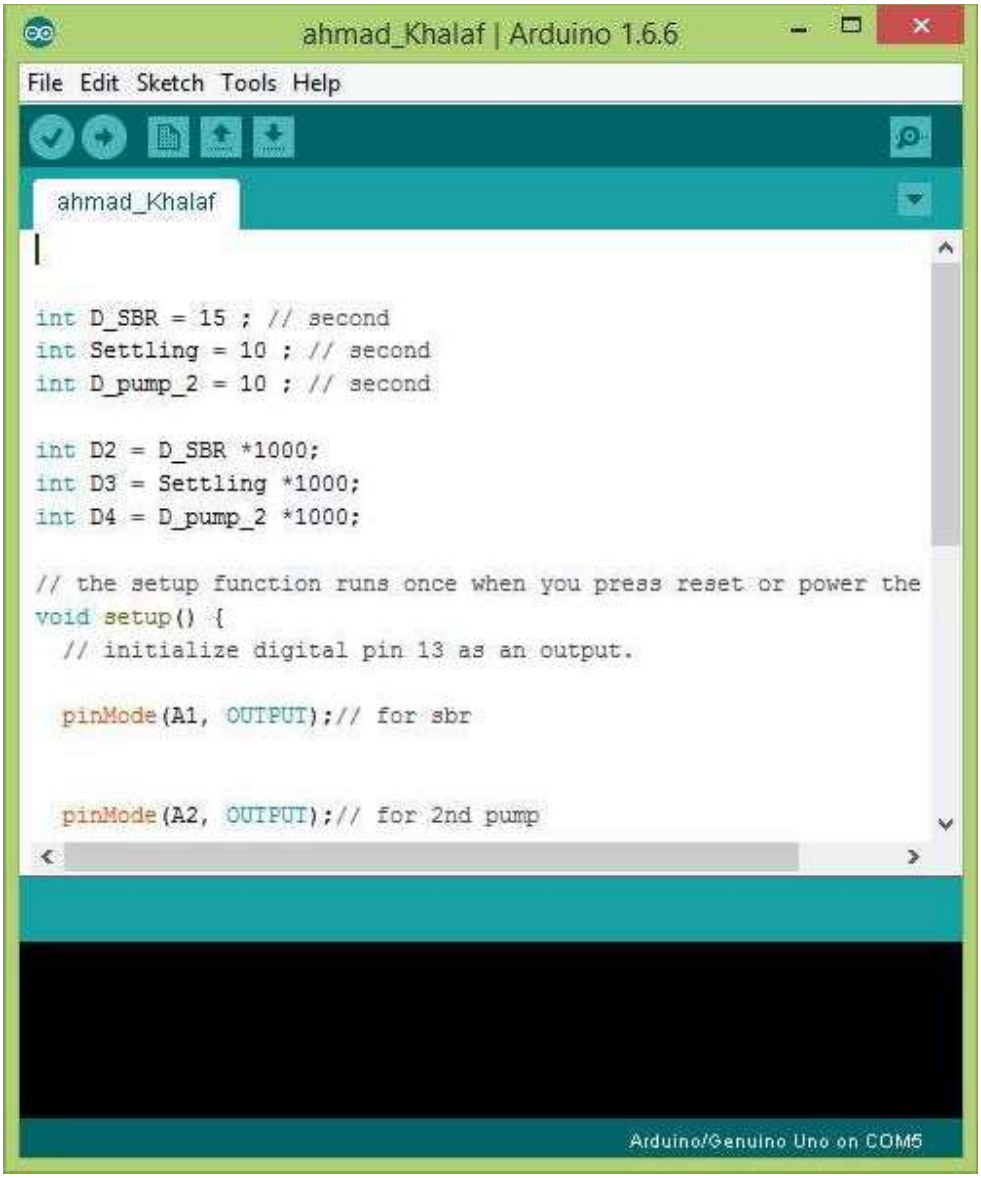
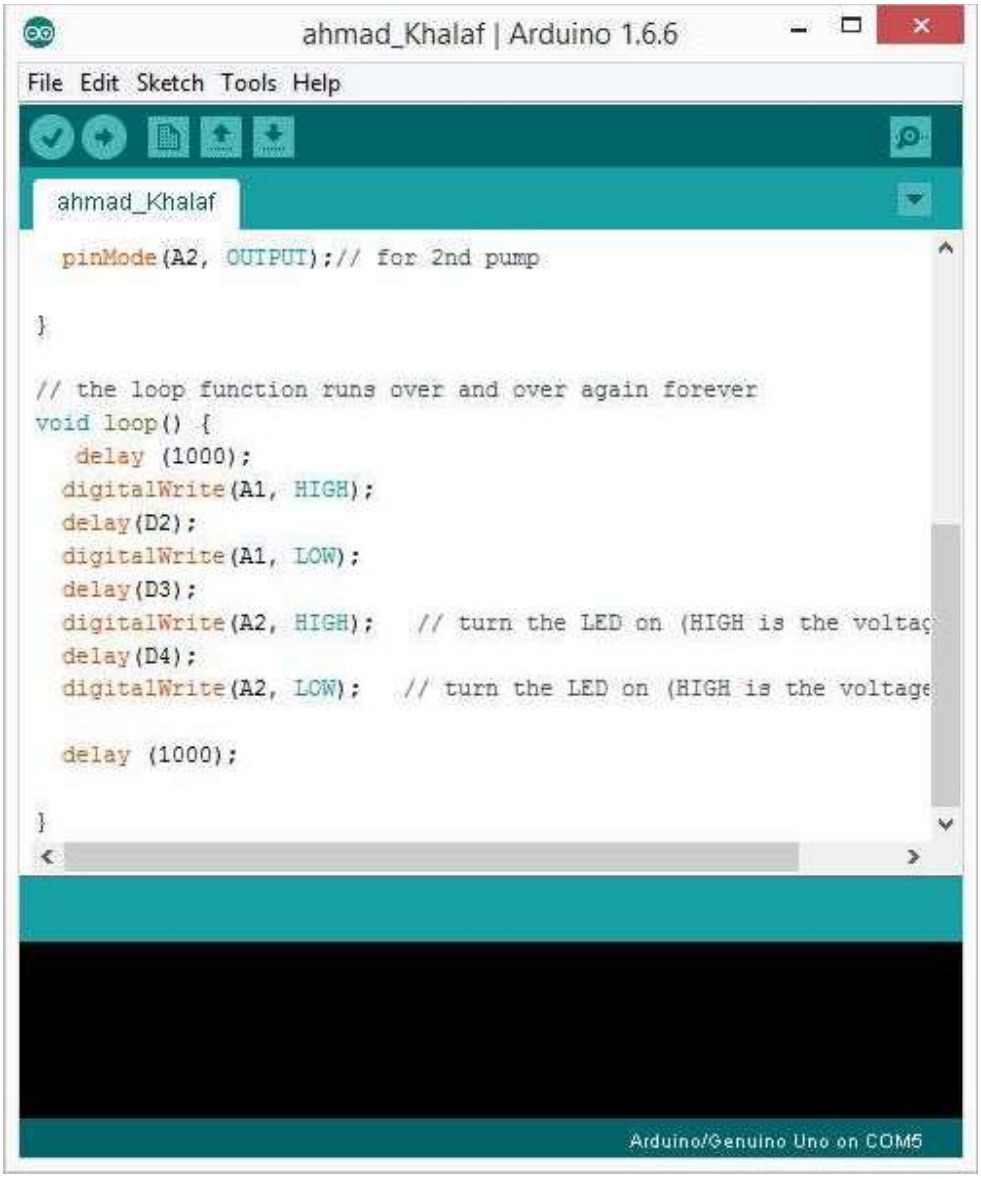


Figure (E-3): Loop Functions and Delays



Appendix F, Experimental Results of SBR Process

Table (F-1): Experimental Results of the Sequencing Batch Reactor SBR Process

Sample ID	pH	Conduct. ms/cm	Turbidity NTU	TSS mg/l	Nitrate mg/l	Ammon. mg/l	D COD mg/l	T COD mg/l	BOD mg/l	DO mg/l	Nitrite mg/l	TN mg/l	Phos. mg/l	TKN mg/l
S1	7.2	4.7	22.3	340.8	14.0	2.4	75.0	104.0	80.0	4.5	0.0	21.5	2.0	7.5
B11	8.1	4.5	17.3	7.1	15.2	0.3	31.0	68.0	15.0	5.6	0.0	15.9	2.3	0.7
B12	8.1	4.7	14.1	6.0	11.0	0.3	44.0	65.0	12.0	5.7	0.0	15.0	2.4	4.0
S2	7.3	4.3	15.5	356.0	50.8	2.7	54.0	87.0	66.0	4.4	0.0	58.6	1.9	7.8
B21	8.0	4.6	8.1	9.9	32.8	0.3	23.0	37.0	10.0	5.5	0.0	33.5	2.7	0.7
S3	7.3	3.9	28.9	295.2	28.8	2.9	68.0	294.0	150.0	4.7	0.0	40.0	2.0	11.2
B31	7.8	3.4	2.3	7.6	14.7	0.3	20.0	33.0	19.0	8.3	0.1	15.2	2.5	0.4
B32	8.0	3.6	3.1	5.1	23.7	0.2	18.0	26.0	20.0	8.5	0.1	24.5	0.5	0.7
B33	8.1	3.7	3.7	4.0	14.0	0.2	25.0	43.0	17.0	8.7	0.0	14.4	1.2	0.4
B34	8.0	3.7	2.1	3.6	15.5	0.2	17.0	29.0	16.0	8.3	0.1	15.9	1.7	0.4
S4	7.4	4.6	20.6	389.0	4.9	3.12	81.0	98.0	70.0	3.6	0.0	15.0	1.3	10.1
B41	7.8	4.3	6.6	9.6	9.9	0.2	39.0	44.0	11.0	6.9	0.0	10.7	2.1	0.8
B42	8.1	4.5	9.7	8.3	10.2	0.1	22.0	38.0	9.0	7.0	0.1	13.4	2.2	3.1
S5	7.7	4.5	45.1	421.5	6.0	2.84	97.0	127.0	90.0	3.5	0.0	17.1	1.0	11.1
B51	8.3	4.7	7.5	9.0	11.4	0.1	31.0	41.0	13.0	6.5	0.1	12.0	2.0	0.6
B52	8.3	4.8	6.4	6.9	11.8	0.2	34.0	51.0	10.0	6.4	0.0	12.5	0.2	0.7
S6	7.7	4.8	36.3	426.6	5.0	2.6	79.0	98.0	80.0	3.1	0.0	17.3	1.7	12.3
B61	8.4	4.4	12.8	10.6	13.8	0.2	37.0	57.0	15.0	4.6	0.0	14.6	1.7	0.8
B62	8.5	4.3	8.0	9.4	13.6	0.3	57.0	77.0	12.0	4.8	0.0	14.1	2.1	0.5

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